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TECHNICAL SESSIONS

APRIL 23, 1930

Morning Session—10:30 A. M. to 12:35 P. M.

REMINISCENCES OF SACRAMENTO, CALIFORNIA

By C. E. Grunsky,* Past-President, Am. Soc. C. E.

"A Civil Engineer's History of Sacramento" might well have been the title of this paper because it dealt with the evolution of that city in all of its civil engineering aspects: Location, river control, municipal engineering, railroading, highways, park reservations, and legislative history.

The first appearance of a town in the neighborhood of Sacramento was named New Helvetia by its founder, Capt. John E. Sutter. It was protected by a fort built in the period, 1840 to 1844. The name, Sacramento, appeared after the gold rush in 1848 and 1849 had begun.

Early History

A significant description of early beginnings was given by Mr. Grunsky, as follows:

"When gold was discovered on the American River by James W. Marshall in 1848, Sacramento had a population of about 300. It had grown to be a thriving city with a population of about 12 000 before the end of 1849. The city was then reached by boat—at first mainly schooners, but later river steamers—and naturally it became the great center of supplies for the mines of the central and northern portions of California.

"A little later when Engineer T. D. Judah made the first surveys for a railroad across the Sierra Nevada Mountains, Sacramento was selected as the starting point and it was here, by the 'big four', all business men of this city, that the plans for the transcontinental line were perfected. * * *

"It was natural, therefore, that the railroad shops should be located at Sacramento. These and the fact that Sacramento was early made the seat of the State Government did much to maintain the city's prosperity in the dull days of the Seventies which followed decreased activities in placer mining."

The seat of government was not at once placed at Sacramento, according to Mr. Grunsky. San José, Vallejo, Sacramento, and Benicia served, successively, until on March 1, 1854, the Legislature met in Sacramento's new Court House.

Mr. Grunsky described the financing problems that were solved before an adequate building was supplied.

*Cons. Engr. (C. E. Grunsky Co.), San Francisco, Calif.

Description of the Site

Sacramento is situated so that river-control problems have been acute from the very beginning. According to Mr. Grunsky:

"Under natural conditions the site upon which Sacramento has been built was subject to inundation at high stages of the American and Sacramento Rivers. Both these rivers, when at flood stages, sent large volumes over bank. The business men and residents had their first serious experience with flood waters in the winter of 1849-50. This was repeated in 1852-53. Thereupon measures were taken to keep the water of American River out of the city and to prevent the Sacramento River from overtopping the water-front street. To this end the water-front street was raised and levees were built along the American River in 1853 and 1854 and along the north bank of Burns Slough—the slough which passed just to the north of Sutter's Fort."

Detailed and thrilling accounts as related by eye-witnesses, depict harrowing experiences that occurred as a result of the floods dating as far back as 1849.

Sacramento After 1879

Mr. Grunsky's personal experience with this city dates from 1879, at a time when,

"The recurrent floods brought the business men of the City of Sacramento to a realization that something more should be done than merely to throw up an earth embankment with the hope that this would confine flood waters to the rivers or would keep them on the river's far side. It was, therefore, decided to raise the down-town streets, particularly J and K Streets about 10 ft. This was done and store and other buildings were modified to adjust upstairs floors to the new street level. Two-story buildings were thus reduced to one-story appearance. For many years after this time, well along into the Eighties of the last century, there were along J and K and the numerous cross streets of the down-town area rows of one-story buildings, generally of brick, in front of which the sidewalks were sheltered by flimsy portico or porch constructions. Here and there only did the business blocks rise to a height of two or three stories above the new street level."

Various civic organizations took up the fight for beautifying and improving the city, including an Improvement Club and the Bric-a-Brac Club. Public agitation resulted in continued civic improvements from that time.

Concerning early attempts at street improvements, Mr. Grunsky stated:

"The first experiments with asphalt paving at Sacramento were made about 1885 or 1886. The material then available was natural, so-called, bituminous rock. This was obtainable at Santa Cruz, where certain beds of sand were permeated with asphaltum of various degrees of hardness. By a judicious mixture of the rock from various parts of the quarries an asphaltic sand was obtainable which had all the valuable properties of the artificial wearing surface produced by mixing sands of various degrees of fineness with asphalt. The use of bituminous rock for paving was meeting with success at San Francisco and elsewhere, why not at Sacramento? And so the experiment was tried on lower K Street where a short stretch of cobble pavement was covered with the bituminous rock. There was no binder course. And, of course, due to varying thickness of the asphalt layer, and aggravated by the thumping of the wheels of the heavy laden, dead-axle, steel-tired trucks of that day, the wearing surface quickly went to pieces and the conclusion was generally accepted that the climate of Sacramento was not suitable for the asphalt pavement.

"Several years later, as Consulting Engineer to the Capitol Park Commissioners, I was called upon to recommend a pavement for the vehicle area in the rear of the Capitol. Despite the prevailing general prejudice, I succeeded in convincing the Commissioners and drew the specifications for a bituminous rock pavement on a 4-in. concrete base. It was prescribed that the concrete should be laid upon the well packed macadam already in place, the surface of which required but little dressing to grade.

"When the first loads of gravel for the concrete were delivered I condemned the material as not suitable. Some of the pebbles were rolled nodules of clay which could be broken with the fingers. 'It is the best obtainable', said the foreman. 'Oh no! Let me show you.' And so a trip was made up the American River 14 miles to Mayhews where excellent material could be had. The work stopped; the superintendent on the job conferred with his principals and a few days later notice was received that aggregate for the concrete would come from Oakland by steamer. Cobble stones there being removed from the streets were put through a crusher and the rock and other good materials resulted in a foundation layer not merely 'good enough', but really first class. Needless to say the pavement came up to expectations and was in service about forty years before any great amount of surface renewal became necessary. It proved that the climate of Sacramento was not at fault."

The civil engineering improvements were properly balanced with improvements in the civic arts. The Sacramento Museum Association, organized about forty-five years ago, has concentrated on the study of natural history and research. Interest in restoring and preserving Sutter's Fort for posterity as an historic point of interest, was first shown in 1889. As an example of how a civil engineer may assist in good works of this kind, the following is quoted from Mr. Grunsky's paper:

"At that time the walls of the Fort were gone. The center building alone was standing. The original south wall of this two-story adobe building had been replaced by a brick wall. Its doors and windows were gone. The structure was at that time occupied, or had only recently been vacated, by a ragpicker or junk-dealer. To the north of the building on the bank of the slough which took its course from east to west past the building was a pile of miscellaneous rubbish, in which the then familiar 5-gal. oil cans predominated."

A party of interested men, including Mr. Grunsky, made a trip to the fort and relocated its exterior walls and the sites of the stores, blacksmith shop, grist mill, and distillery, as they had been during the gold rush. There was little difficulty in doing this, according to Mr. Grunsky, because,

"In connection with litigation some years before, with the aid of Mr. Fairchild, a blacksmith, the southwest corner of the Fort had been acurately located. Mr. Fairchild had operated a blacksmith shop in the Fort and had maintained a coal bin in the corner room. When H Street was filled to official grade at this point the blackened floor of the coal bin was covered by the fill and its outlines were readily disclosed by excavation."

THE CALIFORNIA PLAN FOR CONSERVATION OF WATER RESOURCES

By Edward Hyatt,* M. Am. Soc. C. E.

A paper entitled "The Problem of Maximum Conservation of Water Resources, with Special Reference to the California Plan" was presented by

* State Engr., Sacramento, Calif.

Mr. Hyatt. It was based upon the classification of regions as either arid, semi-arid, or humid. To quote directly:

"Under these criteria Eastern United States is termed humid and the western part arid or semi-arid, the dividing line being approximately at the 99th meridian, which passes somewhat west of the center of Kansas. The State of Kansas has made allowance for the dissimilar water supply conditions within its borders by enacting a different water code for the area west of the 99th meridian from that in force in the eastern half of the State.

"The seventeen States lying partly or wholly west of the 99th meridian have, therefore, come to be considered the arid or irrigation States, which include North Dakota, South Dakota, Kansas, Nebraska, Oklahoma, Texas, and all States west of those named. An association of the State engineers of these seventeen irrigation States is actively functioning on matters pertaining to water, water rights, irrigation, reclamation, and similar items of common interest. These so-called arid States include 60% of the land area of the United States, 50% of the farm lands, and 91% of the total irrigated area, as well as 19% of the total population and 24% of the farm population of the country."

Fundamental Considerations in Water Conservation

Scarcity of water supply makes conservation a more urgent problem in the West than in the Eastern or Middle Western States, said Mr. Hyatt, and:

"If maximum conservation is to be achieved, it naturally follows that the most complete possible utility must be made of existing water supplies for all useful purposes. Before approaching the purely technical phase of the study, therefore, the engineer should have a clear conception of fundamental consideration such as all present and future beneficial uses of water in the region under investigation, their relative importance and amount, whether consumptive or non-consumptive, degree of interference one with another, and legal or commonly accepted priorities, if any."

Uses of Water.—A conservation program must take into consideration all the many and varied uses to which water may be put. Mr. Hyatt presented five broad groups, as follows:

"*First.*—Consumptive, which includes municipal, stock, industrial, irrigation, and some forms of mining and milling.

"*Second.*—The extraction of energy inherent in water by reason of its relative elevation, in which class are hydro-power, including both hydro-electric and hydro-mechanical, and hydraulic and some other forms of mining.

"*Third.*—Use of the buoyancy of water for transportation purposes, consisting of all forms of navigation.

"*Fourth.*—Its utility as a scenic attraction and by reason of fish life maintained. This class consists of recreation and commercial fishing.

"*Fifth.*—The control of water to prevent damage; flood regulation, salinity control, and drainage. Salinity control, perhaps peculiar to California, consists in furnishing fresh water to hold back saline encroachment from the ocean and bays. Drainage is a concomitant of irrigation or of farming wet lands."

All these uses, he declared, are important in California and the effect of interference between the types is an important consideration. Furthermore, the relative legal position of the divergent uses should be known.

Steps in Solving the Problem.—The first step is to collect and analyze pertinent engineering data, especially those relating to the supply of water available. This leads directly to the study of storage, upon which the subject of conservation inevitably hinges. To quote Mr. Hyatt:

"Reservoir sites are often, if not usually, the controlling item in the conservation of a stream's waters. Therefore, reservoir sites are located and searchingly analyzed as to adequacy, cost, and yields through the dry cycles which will determine the economic size and yield. It is not generally possible to capture for use more than 60 to 80% of the mean discharge of a stream, even with unlimited reservoir capacity, which is often further limited by lack of favorable dam and reservoir sites."

Of equal importance with a study of water supply is an analysis of present and future water needs. The principal use made of water should be the guiding consideration in determining its allotment, said Mr. Hyatt. For example:

"In irrigation States, such as California, more than 90% of the water consumptively used is for irrigation. It follows, therefore, that any development program in an arid State will revolve mainly around irrigation, giving due consideration, however, to all other purposes. A determination should be made of the arable lands of the State and of the quantities of water needed for their proper cultivation. This requires a classification of all such lands, which while necessarily of a reconnaissance nature, is based upon soil, topography, climate, location, and upon economic factors."

With these two phases completed, trial studies may be made which will result in a comprehensive plan for utilization. Within the territory considered, some regions will be found with more water than will be needed and other regions with less. Mr. Hyatt explained that then the problem resolved itself into deciding how much water could be economically exported from the one region without detriment to its local demands.

By a proper location of reservoirs, the primary function of the water (irrigation, in California) can be exercised; and, at the same time, secondary functions, such as generation of electricity, flood control, etc., can be accomplished without serious interference. Mr. Hyatt commented on this possibility as follows:

"Flood control and other uses may seem to be difficult of reconciliation in a single storage, due to the fact that to serve flood control alone the reservoir space is reserved for that purpose, while to accomplish the other intents it is ordinarily filled as quickly as possible. Many engineers believe that these uses cannot be combined except through the medium of open ports in the dam which dedicates the storage space above the ports to flood control and the space below to other purposes. An intensive study of flood-flow characteristics, however, has led to the conclusion that the use of reservoirs for flood control is compatible to a marked extent with that for other services.

"Knowing the seasonal characteristics of rainfall and run-off on a given drainage basin and stream, it is possible to design a method of holding a predetermined amount of storage space in reserve through the flood season for equation of the peaks, and after the flood danger has definitely passed to allow the reserved space to fill and permit the reservoir to perform full service thereafter for other uses. While the power head is decreased during the flood season, by passing the larger quantities of water available at that time of year through a lower head, the loss in power output can be kept to a minimum."

The next step recommended was to specify the proper economic order in which the various units should be constructed, since they should not be built more than reasonably in advance of their needs. The chief sources of embarrassment to irrigation projects in the West are due to interest charges on unused works, resulting from optimistic estimates of rates of colonization.

Regarding allocation of costs among all beneficiaries, Mr. Hyatt stated his belief that,

"The State and the United States should be expected to participate in the costs on the bases of State-wide and Nation-wide benefits to be derived under existing State and National policies, or as the same may be reasonably expected to be modified in the future. There will be an undoubted State-wide value in that the cities and industrial areas in non-agricultural regions will benefit from increased markets, commerce, and transportation, and the National Government will be directly aided by whatever is done in the interest of navigation and flood control and also by the reclamation of land."

Finally, he emphasized the legal problems of water conservation as being sometimes insurmountable.

Water Conservation in California

Since 1921, the water problems of California have been studied roughly along these lines. By the end of 1930, said Mr. Hyatt, this investigation will have cost about $1 000 000, exclusive of stream gauging, topographic mapping, and soil surveys.

A vivid description of conditions prior to 1921 is given by Mr. Hyatt, as follows:

"Many areas, large and small, were expanding irrigated areas beyond the dependable water supply, some by over-optimistic estimates based on the records or estimates of flow of a few wet years, but more by pumping from underground sources a greater amount than the replenishment thereto. In one section of the San Joaquin Valley, in the last four years, about 20 000 irrigation pumping plants have drawn from the underground storage an estimated net amount of 2 000 000 acre-ft., in excess of the recharge in the same period. Irrigation diversions on the rivers tributary to San Francisco Bay have so reduced the fresh-water inflow that a delta area of about 400 000 highly productive acres has been seriously threatened by the incursion of salt water from the Bay itself. It is beyond the power either of the people in the San Joaquin Valley and Southern California, whose underground supply is insufficient, or of the delta area, to remedy conditions facing them. These and similar conditions of shortage appeared in a great many places in California between 1900 and 1920, and were accentuated by the series of dry years commencing in 1917."

The present investigation began with a State-wide classification of water and land resources. The next step was to estimate, from available records, the 50-year mean run-off for each stream in the State.

Speaking of available records, Mr. Hyatt declared that there were 277 rainfall stations that supplied records of more than 10 years' duration. Using these, the total rainfall for any one season at any station in percentage of its annual mean, was termed its index of wetness. The succeeding steps may best be described by Mr. Hyatt:

"The indices of wetness for each station for the period were plotted as mass diagrams, showing the accumulated departure from the mean, then by superimposition these diagrams were compared to define the various areas in the State having the same general precipitation characteristics. This established 26 precipitation divisions.

"The State was then divided for purposes of study into 140 drainage basins, each basin being either a major drainage or a group of minor drainages. Stream-flow records were available at 200 regular U. S. Geological Survey stations for varying periods up to a maximum of 28 years, and, in addition, fragmentary records at about 300 additional stations. From these data rainfall run-off curves were platted and by superimposition critically compared both geographically and by types. Through these comparisons the characteristic shapes of the curves both geographically and by type were closely determined and it was possible to extend the run-off curves to streams on which there were no actual flow measurements. By these methods rainfall run-off curves were completed for each of the 140 drainage basins in the State. Applying the indices of seasonal wetness in the 26 precipitation divisions to the rainfall run-off curves in the 140 drainage basins the value of the seasonal run-off for every drainage area in the State for the full 50-year period was estimated.

"Mass diagrams of run-off were drawn for each area. These were platted as accumulated run-off in percentage of variation from the annual mean, thereby producing a graph which gives all the desired information in but a fraction of the space required by the usual mass curves. Irrigation draft lines were then prepared, being platted in identical units and on the same scales as the mass diagrams. Superimposing the draft lines on the mass diagrams, the utilizable flow of the drainage basin with any given storage capacity was obtained, corrected for evaporation."

Then the duty of water was determined for each of the sixteen agricultural divisions. A preliminary plan for the conservation of water resources in California has now been prepared with the purpose of relieving existing emergencies. In summarizing the scope of this plan, Mr. Hyatt stated:

"In Southern California there was prepared the importation of Colorado River water in a manner which is now under intensive study by the Metropolitan Water District of that area. In the Sacramento and San Joaquin Valleys, otherwise known as the Great Valley, the plan contemplated storage of water in the northern water-sheds, surplus to the ultimate future needs of that area, and its transportation to the deficient lands of the San Joaquin Valley several hundred miles to the south. In devising the transmission system a gravity canal was at first considered, but was found to be prohibitive in cost. From a diversion on the Sacramento River at the proper elevation to afford gravity delivery to the southern end of the San Joaquin Valley would require a canal very nearly 1 500 miles in length, which would be in mountain or foothill country for almost its entire distance.

"As a substitute a plan of allowing the water stored in the Sacramento Valley to flow after release down the Sacramento River to the delta area and to be pumped therefrom into the San Joaquin Valley, was investigated and found to present a superior solution. In this set-up the channel of the San Joaquin River would be used as a conduit and by a system of dams and pumping plants the Sacramento River water would be forced up the San Joaquin River against its grade to the desired elevation. This arrangement was found to be vastly cheaper than the gravity lay-out, both in capital and annual costs, and it also had many other advantages, such as more dependable water supply, elasticity of operation, and freedom from legal and water-right difficulties."

Thus, the physical plan has taken fairly definite form. Distinguishing between technical studies and economic studies, Mr. Hyatt declared that, while the former were of greater magnitude, the latter were more difficult. Public interest in the plan is keen and there is a general acceptance of the idea that the conception is sound. The cost estimates vary from $100 000 000 with a minimum number of units to $700 000 000 with a maximum.

In conclusion, Mr. Hyatt declared that,

"If such a plan is carried out under the direction of the State itself (and it does not seem possible for any other agency to successfully accomplish it), other questions of a State or political nature arise. California, climatically and geographically, is separated into several divisions. The northwest coast area with 25% of the water supply of the State and only 2% of the agricultural lands has no water problem and little interest in the situation. The Great Central Valley, with approximately 13 000 000 acres of land and 37 000 000 acre-ft. of mean annual water supply, will require such a plan for its development, and it has furnished the main support for the investigations to date. Southern California, with 20% of the land and only 1% of the water supply, has more than one-half the population and the assessed valuation of the entire State, and is undergoing a water shortage of such severity that relief is imperative.

"It is apparent that a conservation plan, carried out under State auspices, and partly or wholly by State bond issue, must assist all portions of the State in need and not be local in character. It is also apparent that to recommend a plan sound from the engineering and economic standpoints, which will also properly take into account the various sections of the State in such a way as to be satisfactory to the voters of the entire State who must ultimately pass upon it, is indeed a problem of the first magnitude, which can be solved only by the co-operative and constructive efforts of all concerned. Wide vision, sound business judgment, and complete and accurate information are essential to the success of such an undertaking."

DISCUSSION

OUTLINE OF A CONSERVATION POLICY FOR CALIFORNIA

By J. B. Lippincott,* M. Am. Soc. C. E.

This discussion was largely a supplementary contribution to the paper by Mr. Hyatt and laid emphasis on special projects. The gist of Mr. Lippincott's pleas was for California to:

"Support a plan providing for joint co-operation between the State and Federal Governments as well as with local agencies in the building of such extensive and essential works as are of broad importance. These include the conservation of the Sacramento River water, the relief of the San Joaquin Valley, and the provision of an adequate water supply for Southern California from the Colorado River."

Water supply projects for irrigation and domestic service have now been built in California to the limit of local financing. According to Mr. Lippincott:

* Cons. Hydr. Engr., Los Angeles, Calif.

"Those remaining are of major size and cost, such as the Sacramento, San Joaquin, and Colorado River enterprises, which can only be constructed with Federal and State aid in co-operation with local interests."

Since most of the gross revenue from an irrigation agriculture is distributed to labor, transportation, trade, and the professions, agricultural interests should not be required to bear the entire burden of such projects. This should be supported by the community at large, that is, by the State.

Improvements on the Sacramento River.—The estimated 40-year mean annual flow past the City of Sacramento was given by Mr. Lippincott as 24 653 000 acre-ft. This, he pointed out, was comparable to the discharge of the Colorado River, at Yuma, Ariz. (16 000 000 acre-ft.). Reservoirs are needed in the Sacramento Valley if the flood waters are to be conserved for distribution during the period of relatively little flow. To quote Mr. Lippincott directly:

"The saving of the flood waters in reservoirs is not only requisite for the comprehensive service of the Sacramento Valley, but it is also necessary if any surplus is to be available for use in the San Joaquin Valley where it is vitally required. It is not yet proved that there is a surplus above the full requirements of the Sacramento Valley through the critically dry period of years. It has been proposed that a dam may be built on the main river at Kennett, with a capacity of about 3 000 000 acre-ft., which with its hydroelectric plant will cost from $70 000 000 to $80 000 000, depending on the size of the power plant decided upon. * * * The distribution of the regulated water required for the Sacramento Valley should be left to the local interests."

The advantages of this plan of conservation are that: (1) It would reduce flood flows 50% at Red Bluff; (2) aid navigation on the Sacramento River throughout the year; (3) prevent encroachment of salt water into the fertile delta islands; (4) insure an irrigation and domestic water supply for the present requirements of the Sacramento Valley; and (5) possibly, it would leave a surplus for the southern part of the San Joaquin Valley where the supply is deficient.

Water Supply in the San Joaquin Valley.—Mr. Lippincott stated that in the San Joaquin Valley, south of Fresno, Calif., practically all the surface run-off is now utilized. During the growing season thousands of wells are pumped to furnish the necessary water. Furthermore, the towns in the Valley depend upon wells for their domestic supply.

The result is that:

"In the face of these conditions the ground-water levels from Kings River south are falling. In many highly improved localities this is true to so serious and alarming an extent that Federal farm loans have been discontinued. It is not a question of irrigating new areas that is involved, but rather the maintaining of existing developments. This situation can only be met by bringing in a new supply if possible from the Sacramento River Valley to the southern half of the San Joaquin Valley. This region is not able to finance so large an enterprise. Its prosperity is of prime importance to the welfare of the entire State. Its trade goes both to the north and to the south.

"Relief to the San Joaquin Valley can only come through State-wide co-operation. The commercial interests of the Commonwealth cannot afford to let retrogression take the place of growth in a region of such extent, possessing great natural resources but depending on an adequate water supply."

Water Supply in Southern California.—In summarizing the regional aspects of water conservation, Mr. Lippincott said of Southern California that, although it embraces more than one-half the population and wealth of the State, it has only 1% of the State's water supply naturally tributary to it. This estimate, which was accredited to the State Engineer, does not include the Colorado River.

Furthermore, quoting Mr. Lippincott, Southern California's winter floods:

"Are mostly stored in surface or underground reservoirs. Of its water supply 80% is now derived from wells. An aqueduct, 250 miles in length, has been built by the City of Los Angeles from the eastern face of the Sierra Nevada to supplement its meager natural allotment of water. Of its area now served with water 20% is within the boundaries of incorporated towns and this municipal area is rapidly increasing. Its population increased 85% from 1910 to 1920 and this rate of growth is being sustained. Its ground-water supplies are overdrawn and a falling water-plane menaces future development in the regions from San Bernardino to the sea. South of Orange County where ground-water supplies are small, the surface supplies, although now largely conserved in reservoirs, are inadequate to meet the future demands either for domestic or irrigation requirements.

"A new and extensive supply of water is essential to provide for the future growth of Southern California as well as to sustain that which already has occurred. However, one adequate source of additional water is available and that is the Colorado River which is wasting in destructive floods an average of about 14 000 000 acre-ft. into the sea annually, an amount fourteen times the total Southern California water crop south of the Tehachapis and west of the coastal mountains. Adequate storage reservoir sites exist on this stream for the regulation of these waters, which will prevent destructive floods and conserve them for beneficial use. The value of the power produced will largely pay for the cost of conservation storage, but not for the diversion of the water to points west of the Coast Range."

The construction of a dam in Boulder Canyon and the aqueduct from the Colorado River to the coastal plains of Southern California may be expected to improve conditions vastly.

Improvements in the Sacramento Valley.—Mr. Lippincott stated that the Sacramento Valley comprised 3 000 000 acres. The low-water flow of the Sacramento River is practically all diverted for irrigation above the City of Sacramento; but flood flows of 600 000 sec-ft. occur and formerly these overflowed large portions of the Valley and some of the towns.

Improvements involving the expenditure of $57 000 000 have been planned to prevent such overflows and to improve navigation. The costs have been divided equally between the State, the Federal Government, and the Local Reclamation District.

This plan is of value, according to Mr. Lippincott, because,

"It furnishes a precedent for other major projects such as that for the San Joaquin and Colorado River Aqueducts, as well as for the completion of the necessary storage work along the Sacramento."

Water Supply for Los Angeles and Vicinity.—Extensive studies have been made by the City of Los Angeles of the problem of conveying 1 000 000 acre-ft. of water each year from the Colorado River to cities in Southern California. In commenting on the Metropolitan Water District formed to

carry out the project, Mr. Lippincott declared that "such substantial progress has been made by this organization that a change in its existing executive authority would not be acceptable."

As for the program outlined, while it protects the towns, it does not care for the horticultural sections that largely rely on ground-water supplies for irrigation. According to Mr. Lippincott,

"These sources are now being overdrawn and there is little opportunity for their replenishment except from the waters of the Colorado River. While the cities may be able to care for themselves, the rural districts are not able to finance adequately so large a problem. There is a proper sphere for State assistance. There is precedence for such State assistance without direct control in the Sacramento Valley Reclamation Project, referred to, and in the case of the Los Angeles Flood Control District. The Federal Government is now to build the Boulder Canyon Dam. However, there is a flood control charge of $25 000 000 against this project, which under the Act is to be repaid by power sales, which is unusually severe. Federal aid might be extended to this project by relieving it of this burden.

"Adequate engineering data necessary for the adoption of a broad plan for the conservation of the water supply available for the State will be available within the present year (1930). The determination of some proper legal and financial plan remains to be accomplished. The most difficult problem probably will be to convince the cities of the advisability of their assuming their portion of the financial burden involved in a State bond issue. President Hoover, in his letter to the Governor's Conference at Salt Lake City, Utah, August 26, 1929, plainly states that he considers it proper for the Federal Government to build at its expense dams for the regulation of major streams."

HISTORY OF INVESTIGATIONS

By Fred C. Scobey,* M. Am. Soc. C. E.

The first appropriation for the studies outlined by Mr. Hyatt was a grant of $200 000 by the California State Legislature, said Mr. Scobey. This appropriation became available on August 1, 1921, as a result of various movements throughout the State. The late Wilbur F. McClure, M. Am. Soc. C. E., then State Engineer, solicited suggestions from many prominent engineers as to the best procedure for beginning the investigations. Mr. Scobey commented on the sagacity of the replies in the light of later experience.

Two basic questions, according to Mr. Scobey, present themselves to an engineer in traveling over other parts of the West.

"1.—How were the potential agricultural areas of the State determined?

"2.—How were the water requirements for various portions of the State set up?"

Activities prior to the beginning of the present investigation were outlined by Mr. Scobey as follows:

"During the year just prior to the commencement of this investigation various co-operative agencies had contributed funds toward compiling and drafting the Irrigation Map of California. This map showed the 'Irrigated

* Senior Irrig. Engr., U. S. Dept. of Agriculture, Berkeley, Calif.

Areas' to the extent of about 6 000 000 acres. In addition, the 'Agricultural Areas' were depicted under three classifications. These lands aggregated about 17 000 000 acres. Some of this land was being farmed without irrigation and much of it was but desert soil. The estimate of its agricultural possibilities was based wholly on the assumption that it would become irrigated land if a water supply were eventually made available.

"The selection of these agricultural lands was made by the best authorities available at the time. Local studies and soil surveys by the State and Federal Governments and the State University of course yielded the best data. With minor exceptions, the agricultural lands as shown on the map were accepted for the investigation at face value.

"For comparison of data with the Federal Census of Irrigation for the year 1919, statistics of these areas had been assembled by counties. Since the State Water Resources investigation would be conducted in terms of stream basins, some of the first work consisted in re-assembling the local areas in terms of river basins. The net results of this tabulation showed the irrigated areas of 1920 and the unirrigated areas naturally commanded by each major stream. The hydrographic studies of the streams would then determine any surplus or deficiency of water for various assumed degrees of reservoir storage, various degrees of storage being based on various limiting costs of reservoir space. It had long been known that there was a major surplus of water in the Sacramento Valley and a deficiency in the San Joaquin Valley."

The aim of this investigation was to determine the maximum use of water. Therefore, said Mr. Scobey, water wasted or lost by deep percolation on one local area, was assumed to be recovered and used on some lower area. Therefore, the net water requirement on the larger areas approached the absolute consumptive use.

According to Mr. Scobey:

"These assumptions resulted in the determination of net duties for large areas that would not as a rule be applicable to individual projects, even of magnitude. For instance, in a locality where a duty of 2 ft. in depth was determined for a gross area, the net project duty would run from 2½ to 3 ft., measured at the farmer's head-gates. For the gross areas into which the State was divided, the determined duty was, as a rule, somewhat less than the net duty as estimated by various authorities for projects within the same area. However, in a few cases, it was anticipated that the co-ordinated plan would make available more water than had ever been contemplated for project studies and the determined duty allowed more water than authorities had used as a duty that would barely 'get by', on account of inadequate water supply."

These diverse conditions made it impracticable to determine water requirements on a project basis; they were made on the basis of the entire State. Omitting certain desert and mountain areas, the State was divided into "Duty of Water Sections" and a net duty was found for each of them. For example, the entire "floor" of San Joaquin Valley comprises one "section."

Summarizing the progress of investigations to date, Mr. Scobey stated:

"It has now been nearly ten years since the Water Resources investigation started. The agricultural areas and duty of water data were assembled as of the knowledge available in 1921. Likewise, the water yield and flood flows prophesied for the future were based on records up to 1921. Whatever the outcome of the investigations in terms of actual construction of a comprehensive plan, the engineers of the State will always have a co-ordinated yardstick for a comparison with their later data, and engineers everywhere

will watch with the greatest of interest to see the degree of conformity of actual performance to empirical forecast in the items of water yield, reservoir development, water requirements, and ultimate irrigated area."

Afternoon Session—2:10 P. M. to 4:15 P. M.

STATE SUPERVISION OF THE DESIGN AND CONSTRUCTION OF DAMS

By M. C. Hinderlider,* M. Am. Soc. C. E.

In the absence of Mr. Hinderlider, his prepared paper was read by Fred H. Tibbetts, M. Am. Soc. C. E.

"Next to the air he breathes, water is man's greatest necessity; under leash, his efficient servant; uncontrolled, an agent of frightful destructiveness."

This, in a sense, was the text upon which Mr. Hinderlider based his paper. The importance of legislation and the control of rivers, he said, was recognized in early civilizations of Babylonia, Assyria, Egypt, and Arabia. Succeeding generations have added to this sense of importance until, to-day,

"Practically all nations are alive to the economic value of systematic control and utilization of this natural asset, not only for consumptive purposes, but in the interest of soil conservation, inland navigation, irrigation, power, and flood control."

Mr. Hinderlider cited the investigation of proposed flood-control projects along the Mississippi as evidence of this interest. Such large projects will involve the creation of great storage basins above intensely populated and highly developed areas. Hence, the subject of safety is of paramount concern.

The Dam as a Potential Menace

Quoting Mr. Hinderlider directly on this subject:

"Every dam, regardless of its size, is in some degree a potential menace to everything below it. There is nothing so relentless in its immediate destructiveness, so uncontrollable and deadly as huge volumes of water suddenly released. The effects of cyclones, earthquakes, and even volcanoes are generally local, and their occurrence is infrequent, and such menaces usually furnish warning of their approach. Great conflagrations are subject to control by modern methods, and science has developed means for successfully combating the ravages of diseases and epidemics; but no means will ever be contrived for overcoming the ruthless destructiveness of huge bodies of water suddenly released from restraint. * * *.

"The number of recorded failures of dams, however, will doubtless compare favorably with the failures of other engineering structures comparable in magnitude, and in menace to life and property. Such failures, however, have been all too frequent, and constitute a reflection upon the ability of the engineering fraternity. Fortunately, or unfortunately, the psychological result of such failures is of short duration, the effects soon forgotten until the tragedy is re-enacted."

* State Engr. of Colorado, Denver, Colo.

The paper contained a valuable detailed list of dam failures, compiled by R. J. Tipton, Assoc. M. Am. Soc. C. E., under the direction of the author. Only a summary list of these failures can be published at the present time.

Commenting on this tabulation, Mr. Hinderlider stated that,

"An analysis of this compilation of data discloses that inadequate spillway facilities and defects in outlet works were the causes of most failures of earth dams, while foundation troubles were the principal causes contributing to the failure of masonry dams, or earth dams with masonry cores."

As might be expected, Mr. Hinderlider reported the largest number of failures in the States that have the largest number of dams in operation. Colorado, for example, which has about 1 000 dams under the supervision of the State Engineer, has been a pioneer in the construction of dams and this has afforded ample opportunity for mistakes. Happily, however, as pointed out by Mr. Hinderlider, only one small earth dam, 17 ft. high, has failed in that State in the past seven years.

Basic Need for Supervision

While it is true that "every dam is in some degree a menace to life and property," yet, said Mr. Hinderlider,

"These structures are an indispensable factor having to do with the welfare of the people of all countries. As such, their safety is a matter of such concern to the general public as to bring their supervision properly within the scope of the police prerogatives inherent in all civilized nations."

Another important adjunct to this paper was a digest of the laws of all the States and of the principal countries of the world, pertaining to dam design. Mr. Hinderlider analyzed these abstracts and offered the following comment:

"This police power or authority is defined as 'the power vested in the legislative body to make such laws as it shall judge to be for the good of the Commonwealth and its subjects. This power governs men and things extending to the protection of lives, limbs, health, comfort, and quiet of all persons and for the protection of all property within the State.' The exercise of this power of a State or Nation to protect the health and lives of its citizens from all sorts of evils and dangers has spread rapidly during the last fifty years.

"Such police powers are very broad, as they must necessarily be in order that the objective sought shall be attained. Such protection is thrown around the citizens of practically all civilized countries and extends to factories, mines, packing plants, dispensaries, transportation systems, buildings and structures for whatever purpose used, and within comparatively recent times, to lines of agricultural development, food supplies, and for the control and eradication of contagious and infectious diseases, and, in fact, wherever the safety and well-being of the citizen and his property is beyond his individual ability to insure. Such supervision on the part of a State is the gradual outgrowth resulting from increase in population and the complexities incident to the development of civilizations the world over. It is a function of Government which is becoming increasingly great and of apparent necessity in the evolution of the social, economic, and commercial development of a nation. Its beneficent aspects are readily recognized, although having the effect of subordinating individual liberty in the greater interest of public welfare."

Supervision in some degree is exercised in most of the States, and centralized control is being more and more recognized as a public necessity.

PARTIAL LIST OF DAM FAILURES WITH CAUSES, 1799–1929 INCLUSIVE.

(Compiled April, 1930.)

CAUSES OF DAM FAILURES ARRANGED IN ORDER OF NUMBER OF OCCURRENCES.

Reference No.	Cause.	Reference No.	Cause.
1	Inadequate spillway.	8	Ice pressure or disintegrating effect of.
1(*a*)	Inadequate spillway. Overtopping by flood wave due to failure of dam above.	9	Improper operation or inadequate maintenance.
2	Inadequate cut-offs. Porous foundation allowing leakage and erosion under earth dams, and for sliding in rigid types.	10	Burrowing rodents.
3	Faulty construction; material not properly compacted in earth dams; poor construction in masonry dams.	11	Poor materials.
4	Inadequate cut-offs around conduits in earth and rock-fill dams.	12	Unstable or structurally weak foundation.
5	Faulty design of section; slopes too steep in earth dams; section too light in masonry dams.	13	Conduits through earth or rock-fill dams not properly supported to prevent settlement or failure. Improper location of valves.
6	Inadequate means for stream control during construction.	14	Insufficient provision against erosion from back-wash below dam or spillway.
7	Improper use of clay in earth and hydraulic-fill dams.	15	Earthquakes.
		16	Miscellaneous and undetermined.
		17	Failure of bottom in small water-works reservoirs.

SUMMATION OF DAM FAILURES FROM DETAILED LIST WHICH FOLLOWS.*

Type of dam.	Height, in feet.	NUMBER OF FAILURES UNDER EACH CAUSE. REFERENCE NUMBER OF CAUSE REFERS TO ABOVE LIST.																		Total, percentage.	
		1.	1*a*.	2.	3.	4.	5.	6.	7.	8.	9.	10.	11.	12.	13.	14.	15.	16.	17.		
Earth	0– 25	13	3	5	1	4	......	......	......	......	......	......	......	......	......	1	......	2	......	29	(22½)
	25– 50	14	1	3	......	5	2	2	2	......	1	......	1	......	......	1	1	1	......	34	(26½)
	50– 75	3	......	5	4	2	2	2	1	......	......	1	1	......	......	1	......	......	......	22	(17)
	75–100	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	1	......	1	(1)
	100–125	......	......	......	2	......	1	1	......	......	......	......	......	......	......	......	......	......	...	3	(2½)
	125–150	......	......	......	......	......	......	......	......	......	......	......	......	1	1	......	......	......	......	2	(1½)
	150+	......	......	......	1	......	......	......	2	......	......	......	......	......	......	......	......	1	......	4	(3)
	Not given	8	2	4	1	4	......	......	1	......	1	1	2	.....	1	......	......	2	5	33	(26)
Total		38	6	17	9	15	5	5	6	......	2	2	4	1	2	3	1	7	5	128	......
Percentage		(30)	(4½)	(13)	(7)	(12)	(4)	(4)	(5)	......	(1½)	(1½)	(3)	(1)	(1½)	(2)	(1)	(5)	(4)	(100)	......

* Numbers in parentheses denote percentages of the total in each case.

PARTIAL LIST OF DAM FAILURES WITH CAUSES.—(*Continued.*)

Type of dam.	Height, in feet.	Number of Failures Under Each Cause. Reference Number of Cause Refers to List.																		Total, percentage.	
		1.	1*a*.	2.	3.	4.	5.	6.	7.	8.	9.	10.	11.	12.	13.	14.	15.	16.	17.		
Rock-fill	0–25			1																1	(10)
	25–50	2										1								3	(30)
	50–75				1															1	(10)
	75–100																			0	(0)
	100–125	1			1															2	(20)
	125–150	1																		1	(1)
	150+							1							1					2	(2)
Total		4		1	2			1				1			1					10	
Percentage		(40)		(10)	(20)			(10)				(10)			(10)					(100)	
Masonry gravity type	0–25			6	2		2			4	1							1		16	(32)
	25–50	3	1	3	6		4			1	1	1								20	(40)
	50–75			2	1															3	(6)
	75–1 0	1																		1	(2)
	100–125	2																		2	(4)
	125–150						1													1	(2)
	150–200			1																1	(2)
	200+							1												1	(2)
	Not given			4										1						5	(10)
Total		6	1	16	9		7	1		5	2	1		1				1		50	
Percentage		(12)	(2)	(32)	(18)		(14)	(2)		(10)	(4)	(2)		(2)				(2)		(100)	

PARTIAL LIST OF DAM FAILURES WITH CAUSES.—(*Continued.*)

Type of dam.	Height, in feet.	Number of Failures Under Each Cause. Reference Number of Cause Refers to List. 1.	1a.	2.	3.	4.	5.	6.	7.	8.	9.	10.	11.	12.	13.	14.	15.	16.	17.	Total, percentage.	
Arch—single and multiple	50-100				1					1				2						4	(67)
	100-150				1															1	(16)
	150+						1													1	(17)
Total					2		1			1				2						6	
Percentage					(33)		(17)			(17)				(33)						(100)	
Reinforced concrete. Ambursen type	15 to 51			6 (100)																6 (100)	
Steel	70			1 (100)																1 (100)	
Timber	1 dam, 100; 7 dams, 12-36	2 (25)		4 (50)	1 (12)						1 (13)									8 (100)	
Grand total		50	14	38	23	15	13	7	6	6	5	4	4	4	3	3	1	8	5	209	
Percentage		(24)	(7)	(18)	(11)	(7)	(6)	(4)	(3)	(3)	(2)	(2)	(2)	(2)	(1)	(1)		(4)	(2)	(100)	

Nature of Supervision

No one man or small group should be given authority to pass judgment on all phases of dam design and construction, because,

"Regardless of the extent of the study and experience of the individual engineer or geologist, each has had different problems to meet and overcome in traveling the rough road of a professional career. It is the well co-ordinated results of such schooling that contributes in the greatest degree to the safety of the public. The conditions surrounding the design, location, and construction of every dam, has its own peculiarities which should be studied, analyzed, and weighed from all points of view and under all the probable conditions under which the structure is likely to function. It is rarely possible to find in a single individual a combination of experience involving full knowledge of design, construction methods and materials, geology, hydrography, effects of temperature changes, disintegration, and the many other elements which contribute to the safety or weakness of a dam."

In Colorado, the supervising authorities not only suggest, quoting Mr. Hinderlider:

"The type of structure as indicated by the site and geological formation, run-off conditions, availability of construction materials, etc., but for the purpose of conserving time and expense of the owner, it is recommended to the engineer that a preliminary draft of his plans and specifications be submitted for criticism prior to the preparation of the final draft for approval. This plan has seemed to work well in practice."

Nevertheless, quoting further:

"The responsibility resting upon a public official in charge of dams is indeed a heavy one, not only as a result of his approval of the plans for such structures, but because of possible internal weaknesses in dams with which he may have had no former connection. Fortunately, as in the case of human diseases, certain symptoms usually warn the careful observer of trouble ahead, and therein lies one of the best arguments for centralized responsible public supervision over such structures, which enables some one to act promptly and with authority."

The District of Columbia and the States of Delaware, Louisiana, Minnesota, Nebraska, and Oklahoma are the only ones that exercise no public supervision over dams, according to Mr. Hinderlider. In the remainder of the States:

"Such supervision ranges from that relating to damages which may result from the action of back-water above a dam, the imposing of regulations in the interest of fish propagation, navigation, and sanitation, to the higher purposes of protection to life, property, and industries which may be placed in jeopardy by such structures. Such supervision is vested in various agencies, ranging from Courts, municipalities, commissions, and counties to the State, and departments of the National Government. As is to be expected, the degree of such regulation increases with the need for dams in the economic and commercial development of such State or country. In many of the States where the topography of the country and gradient of the streams are conducive to power development in only a minor way, the need for public regulation is not so urgent, while in those States and counties where conditions and demands are favorable to the greatest utilization of their water resources, will generally be found the greatest degree of public supervision."

In many of the States, supervision relates not so much to safety as it does to matters of navigation, power development, stream pollution, and flood control. According to Mr. Hinderlider:

"An analysis of the laws of all the States discloses that inspection and supervision over dams in the interest of public safety is exercised in but nineteen. Of these nineteen States, fourteen have regulatory laws which may be considered effective or reasonably so. In only twelve of the States may existing laws relating to State supervision be considered ample for the safeguarding of the public's interest where dams of considerable magnitude are contemplated. In more than one-half of all the States there are either no laws relating to public control of dams which may be considered a menace to life and property, or such laws as exist are for the minor purposes heretofore mentioned. Accordingly, it would appear desirable that in such States careful study of this subject of public supervision of dams should be made. Practically all States of the Rocky Mountain region and the Pacific Coast, have quite effective laws relating to this most important matter. This condition also applies to several of the Atlantic seaboard States, such as New York, New Jersey, Pennsylvania, Maine, New Hampshire, Vermont, Connecticut, and Rhode Island."

The Engineer's Responsibility as Regards Legislation

The author urged the engineer as a specially qualified, public-spirited citizen to take an active part in supporting worthy legislation along these lines, because,

"It is the engineer trained in the theory of stresses and strains and the characteristics and frailties of the materials with which he must work, who is fully appreciative of the real magnitude of such undertakings and of the tremendous responsibilities they represent. Activity on the part of engineers, who by study and experience are justified to speak on this important subject, is a public service which the engineer could, and should, render. No one is so qualified as he, to point out the dangers and to indicate the way.

"Some of the disadvantages resulting from public supervision over the design and construction of dams are attributable to the fact that much of such legislation was enacted without adequate information, or is the result of sporadic attempts to correct, from time to time, deficiencies in previous legislation. This has frequently resulted in contradictions, ambiguities, or complete lack of some essential, and too often has resulted in the objectionable features of divided authority. Consequently, the laws of many States relating to this most important subject are a 'hodge podge,' making for inefficiency in their administration, with the resultant unwarranted sense of security which the public has every right to expect under State supervision. The success of any law depends largely upon the degree of public approval back of it, and upon the confidence reposed in the official charged with the duty of administration.

"It is also apparent that the beneficial effects of any law are measured by the degree of efficiency with which such law may be executed or administered and, regardless of the safeguard which a law may seek to throw around the design and building of such important structures as dams, the effectiveness of such efforts will depend very largely upon its enforcement. It is just as apparent also that the actions of a public official are circumscribed by the limitations and weaknesses of the law which he is charged with enforcing. The first requisite, therefore, are laws as nearly ideal as may be, for the attainment of the desired objective. Hence, the desirability that laws relating to the design and construction of dams should reflect the best engineering thought and experience."

Codes for Supervision of Dams

While constitutional provisions in existing statutes may make the application of a general code difficult, if not impossible, in many States, Mr. Hinderlider expressed his belief that,

"Such a code would be invaluable as a background for the framing of needful legislation applicable to conditions in any particular State or country."

He felt that present knowledge was sufficient to justify the formulation of such a code as a guide to the framing of proper legislation if for no other reason. To quote:

"Within recent years, and as a result of intensive studies by the constituent bodies of Engineering Foundation and other similar organizations, uniformity of practice and procedure with respect to building materials, many types of structures, and related matters has been adopted and recognized in almost all lines of engineering endeavor. A code of requirements relating to the art of dam building, especially if endorsed by the Society, would be invaluable in the drafting of legislation providing for public supervision over the design and construction of dams."

Especially, Mr. Hinderlider commended the work of Fred A. Noetzli, M. Am. Soc. C. E., for his contribution on this subject.* The recently passed Act of the last Legislature of California was also cited as a model of its kind. This was largely the result of crystallized public opinion following the St. Francis Dam disaster.

Provision for State Supervision

Mr. Hinderlider presented ten "do's" and five "don'ts" as guides to providing for State supervision. These are quoted in their entirety as follows:

"State supervision of dams should include authority:

"1.—To pass upon the design, construction, maintenance, and operation of all dams and reservoirs of a certain minimum height and capacity.

"2.—To require, preliminary to commencement of construction, complete data on the geology, topography, rainfall, and run-off characteristics of the drainage basin, samples of materials from foundations and those to be used in construction, and all other information needful for a proper determination of the suitability of the dam site.

"3.—To require complete plans, specifications, and analyses of design, and also authority to modify such plans and specifications prior to and during the progress of construction when conditions would seem to justify such changes.

"4.—To require continuous inspection of new construction and all materials entering into the same, preferably at the expense of the State.

"5.—To employ consulting engineers and geologists either upon the authority of the State or upon the request of the owner, such authority to apply not only to contemplated construction and dams under construction but also to all existing dams.

"6.—To exercise supervisory control during construction and operation of the dam following construction.

"7.—To regulate at all times the amount of storage back of any dam.

* *Western Construction News*, September, 1929.

"8.—To require all dams to be maintained in a safe condition.

"9.—For the administrative official to enforce all provisions of the law with respect to the construction, operation, and maintenance of dams, with recourse to the Courts only under exceptional conditions which would require a prompt determination of the matter by the Court.

"10.—For the provision of ample funds to carry out State supervision as herein provided.

"State supervision should not include:

"1.—Financial obligation on the part of the State in case of failures.

"2.—Imposition of unnecessarily drastic regulation not essential for safety.

"3.—Unnecessary interference with methods of construction.

"4.—Authority to assume the functions of the practicing engineer with respect to the preparation of plans and specifications, and related data in connection with the design and construction of a dam.

"5.—Authority for State supervision in too minute detail."

The last "don't" was further explained thus:

"State supervision should be limited to the approval or disapproval of the plans and data presented, and the authority to require essential amendments thereto, which, in the judgment of the State official, appear to be needful for insuring the proper degree of safety."

Advantages and Disadvantages of State Control

State control of the design, construction, and maintenance of dams has the following advantages, in the opinion of Mr. Hinderlider:

"(*a*) Centralized authority with its singleness of purpose and responsibility to the public.

"(*b*) Minimum of effects of local and political influences.

"(*c*) Prompt and concerted action in anticipation of dangers and the forestalling of failures under most conditions.

"(*d*) Uniformity of procedure in design and inspection.

"(*e*) Greater assurance of disinterested inspection.

"(*f*) Co-ordinated control in administration of the uses of stored water and stream flow, especially in arid regions.

"(*g*) Continuity and permanency of records and public accessibility to same."

On the other hand, the following disadvantages were offered:

"(*a*) Danger of incompetence of, or prejudice on part of, State official.

"(*b*) Lessening of the sense of responsibility and precaution on part of owner.

"(*c*) Possibility of injustices to owner resulting from unwarranted requirements of State official.

"(*d*) Erroneous assumption on the part of property owners below a dam that State supervision is tantamount to State responsibility in a pecuniary way.

"(*e*) Dangers accompanying increase in bureaucratic authority."

Qualifications of Supervising Officials

In his closing remarks, Mr. Hinderlider offered a set of twelve pertinent specifications as a guide in selecting supervising officials, quoting,

"The qualifications of the official charged with the important duties of State supervision of dams should include:

"1.—An understanding of the fundamental principles affecting the stability of all types of dams. This requirement, in general, presupposes that he is a technical graduate or from previous preparation is competent to understand the mathematics of engineering design.

"2.—An appreciation of the advantages of the science of geology and its adaptation to the art of dam building. He should also be thoroughly grounded in the knowledge of climatology and hydrography.

"3.—Broad experience in the problems of construction, which is conducive to a discriminative sense of values and dangers, and of materials, their properties, and uses.

"4.—A willingness to improve his knowledge of the art by consulting with and studying the methods and theories of others. He should be imbued with an honest desire to learn the lessons taught by the failures of the past.

"5.—He should have an exalted sense of his duty to the public and the ability to visualize the grave responsibilities with which he is charged.

"6.—Complete freedom from any influences other than the motivating ones of his official duties.

"7.—Tenacity of purpose.

"8.—An appreciation of the value of systematic methods in the conduct of his office and the essential need for reliable data at all times.

"9.—Progressive ideas, albeit leavened with that sense of responsibility which his duties impose.

"10.—The confidence of his fellow engineers and the public.

"11.—A genuine desire to co-operate, but the stability of purpose not to allow his enthusiasm to get the better of his sober judgment.

"12.—Finally, that rare combination of natural ability and experience commonly known as 'horse sense.'"

DISCUSSION

PUBLIC SAFETY AND STATE SUPERVISION OF DAMS

By H. W. Dennis,* M. Am. Soc. C. E.

That Mr. Hinderlider presented a strong case to support his statement that dams are potential sources of danger, was pointed out by Mr. Dennis. He concurred in the idea that dams should be designed with the utmost skill and care and that the designs should be reviewed by some independent agency.

Continual Need of Research.—As a subject for discussion, Mr. Dennis quoted as follows from the original paper:

"Recent study and practice relating to the design of practically all conservative types of dams have resulted in the development of principles generally recognized as being sound, although there is still some diversity of opinion with respect to important theories affecting the safety of huge structures of the gravity and lighter than gravity types of masonry dams.

"This is to be expected since much remains to be learned concerning the physical and chemical properties of building materials, their behavior under combinations of loading, temperature changes, chemical reactions, and variations in construction details, all of which are impossible of complete control."

* Chf. Civ. Engr., Southern California Edison Co., Los Angeles, Calif.

This quotation, he said, with the list of failures and their causes, indicates that there are still many things concerning the design and construction of dams that the engineer does not know.

Recounting the difficulties encountered by Engineering Foundation in raising funds for the Stevenson Creek Dam Investigation, Mr. Dennis argued that it should be a function of the State, as a part of its supervisory policy, to provide funds for scientific research beyond the power of smaller organizations to promote.

State Supervision and the Police Power.—Since "effective legislation in connection with this subject seems to have required the invocation of the theory of the police power of the State"—that is, supervision in the interest of public safety—Mr. Dennis was inclined to question the supervisory recommendations made by Mr. Hinderlider that did not have public safety as their underlying principle. Quoting:

"I am willing to accept many of these recommendations which he makes, if there is added after each one, 'so far as the public safely is involved,' but I cannot agree with the author that the supervisory body should be given blanket authority to exercise control over these things regardless of whether they involve the public safety or not."

The most important clause in the paper by Mr. Hinderlider (according to Mr. Dennis) was that,

"State supervision should be limited to the approval or disapproval of the plans and data presented and the authority to require essential amendments thereto, which, in the judgment of the State official, appear to be needful or insuring the proper degree of safety."

THE IMPORTANCE OF PERSONNEL

By N. A. Eckart,* M. Am. Soc. C. E.

The discussion of Mr. Hinderlider's paper, prepared by Mr. Eckart, was presented by G. W. Pracy, M. Am. Soc. C. E.

Mr. Eckart commended the paper in that it set forth concisely, the author's idea of the advantages and disadvantages of State supervision and also the qualifications that the official charged with supervisory control should possess.

Referring then to the question as to who shall be responsible for supervision, Mr. Eckart declared that this will not be decided by the Engineering Profession because,

"The right of the State to vest supervision and control of dams in the State Engineer can not be questioned as a part of its police power looking to the public safety."

The form of legislation necessary to accomplish this end, however, will be a subject of deep concern to the profession. There is another phase of the subject that is even more important to the profession. Quoting Mr. Eckart:

"The practising engineers are interested in the individual who is chosen to exercise this power, not only as to his qualifications from the viewpoint of his training and technical ability, but even more as to his executive ability and the broadness of his view and the soundness of his judgment, or, as the author

* Gen. Mgr., San Francisco Water Dept., San Francisco, Calif.

terms it, the amount of his 'horse sense'. The right man in charge of State supervision can make almost any legislation workable and secure the desired results in soundness of design and construction, with the minimum of interference with the work of competent engineers. The wrong man can play havoc with the best engineering plans and construction methods and program; petty interference, incompetence, indecision, or delay in approving proper plans or foundations may add hundreds of thousands of dollars to the cost of a project, as the delay of a few weeks at some stage may result in throwing the work into the flood period and possibly cause the loss of a season's run-off."

The importance of this feature was further emphasized by Mr. Eckart in the statement that,

"The greatest advantage of State supervision is in insuring disinterested and independent review of all dam design and foundation conditions and independent inspection during construction. The value of this independent check is dependent on the competence of the reviewer. If incompetent the review is more than worthless, for in many cases it will result in preventing a competent review and check by independent consulting engineers and geologists who otherwise would have been called in. It behooves engineers to use their best endeavors to have the high standard of the officials maintained."

* * * * * * *

"As to the qualifications of the official charged with State supervision of dams, added to those which the author has specified, the official should have a good fighting heart, the ability to stand on his feet and forcibly and clearly express himself to defend his views, and above all he should have the moral courage to accept full responsibility for the failures as well as the successes in his administration of the office."

The Code of Practice.—Mr. Eckart recommended the adoption and promulgation of a Code of Practice by State Departments so as to avoid much delay and unnecessary work in re-design and re-submission of plans. Specifically, he urged that,

"The Society, either through one of its Sections, or a committee, should take the lead in developing a model draft of legislation covering State supervision of dams, including a code of practice, which should result in greater uniformity and practicability of such legislation and procedure throughout the country.

Co-Ordinated Control Will Not Necessarily Follow from State Supervision.—"The writer can not agree with the author as to all the advantages claimed for State control. There is no assurance, for instance, of the minimum effects of local and political influence; this is no doubt true as between county and State control, but it does not hold between State control and absence of same. Co-ordinated control in administration of the uses of stored water and stream flow does not necessarily go with the State supervision of dams. Such power might be considered as an advantage for the general public interest, but might result in serious detriment to the development of the resources of a State; it would depend largely on how administered."

RESUME OF DAM SUPERVISION IN CALIFORNIA

By G. W. Hawley,* M. Am. Soc. C. E.

Before the enactment of legislation to govern the supervision of dams in California there were several regulatory agencies with varying degrees

* Deputy State Engr. in Chg. of Dams, Sacramento, Calif.

of authority and responsibility exercising jurisdiction in this respect. After the failure of the St. Francis Dam, quoting Mr. Hawley, "public opinion was crystallized in a unified and widespread demand for co-ordinated and authoritative supervision of dams."

The present California law (Chapter 766, Statutes of 1929) was the result. The following quotation in the decision rendered Bent Brothers *vs.* Campbell District, by the State Court of Appeals, substantiates that law:

"'The police power is an attribute of sovereignty, and exists without any reservation in the Constitution, being founded upon the duty of the State to protect its citizens and provide for the safety and good order of society. It corresponds to the right of self-preservation in the individual, and is an essential element in orderly government. * * * It has for its object the improvement of social and economic conditions affecting the community at large, and, collectively, with the view of bringing about the greatest good to the greatest number. On it depends the security of society, order, the life and health of the citizen, the comfort of existence, the enjoyment of private life, and beneficial use of property.'

"'That the police power of the State to supervise and regulate the construction and maintenance of dams impounding large bodies of water, remained unexercised until the disastrous consequences following the breaking of the St. Francis Dam in the southern part of the State, is no argument against its existence, but the experiences attending the breaking of that dam emphasize the necessity for, and the constitutionality of, the police powers being extended to, and including, such structures in order that the safety of persons and property may be conserved. With these statements as a premise, we think the conclusion clearly follows that the Act of the Legislature approved June 10, 1929, is constitutional in all its essential provisions, as not only a proper, but as a necessary, exercise of the police powers of the State'."

Scope of the California Law.—Mr. Hawley outlined the scope of the law briefly as follows:

"The law places under the jurisdiction of the State Engineer all dams in California, other than Federal dams, which have a capacity of 10 acre-ft. or more, or a height of 15 ft. or more, regardless of ownership or supervisory control. Provision is made in the Act whereby the State Engineer is authorized to co-operate with agencies having joint jurisdiction, such as the California Débris Commission, Federal Power Commission, and U. S. Forest Service. The Department is invested with authority under the police power of the State and directed to supervise the construction, enlargement, alteration, repair, maintenance, operation and removal of dams for the protection of life and property. All dams in the State whether heretofore or hereafter built or under construction at the date the legislation was effected are under the jurisdiction of the State Engineer and application for approval of same must be filed.

"Every owner of a dam completed prior to the effective date of the Act is required to file an application for approval of the dam, this application to be accompanied by such available and appropriate information concerning the dam as may be required by the Department.

"Subsequent to the effective date of the Act the construction of any new dam or the enlargement, repair, or alteration of any dam shall not be commenced until the owner has applied for and obtained from the Department written approval of the plans and specifications. It is required that the application for approval of plans and specifications for a new dam shall set forth the location, type, size, and height of proposed dam and appurtenant works; contemplated use and storage capacity of the reservoir and such other pertinent

data as the Department may require concerning foundation conditions, drainage basin area, precipitation, flood flow, and other appropriate data. It is also required that a filing fee, based on the estimated cost, shall accompany the application. The State Engineer is empowered to approve or disapprove application or require modification or revision of any incomplete, defective, or insufficient application.

"During the construction, enlargement, repair, or alteration of any dam the Department is required to make or cause to be made such continuous or periodical inspections, investigations, or examinations as may be necessary to secure conformity with approved plans and specifications; and in order to insure safety, the State Engineer has authority to order revisions or modifications in the plans and specifications, or, if conditions are revealed which will not permit the construction of a safe dam, the approval may be refused or revoked."

A certificate of approval is issued by the State Engineer when the dam is completed in accordance with plans and specifications.

Administration of the Act.—Since this Act was passed on August 14, 1929, the Division of Water Resources has been actively engaged in carrying out its provisions. According to Mr. Hawley, there are now about 650 dams under its jurisdiction with as many as 1 000 other sites available for future development. These may vary in height from 15 to more than 350 ft., and comprise all the well recognized types. They are situated at elevations from sea level to 11 500 ft. and they empound from 10 to 1 300 000 acre-ft., with spillway capacities of as much as 128 000 sec-ft.

Mr. Hawley described the difficulties of gathering data on many dams built in former years, so that certificates of approval could be duly issued. In his own words:

"To accomplish the desired objective, namely, the determination and establishment of safety of each of these 650 existing dams, in addition to supervising new construction, an experienced and sufficient personnel is being organized to cope with the many involved technical and practical problems. The activities of the Department are grouped in six general classifications. Hydrographic studies, geological examinations, stress and structural analyses, supervision during construction, field investigation and examination of existing dams and appurtenant works, and supervision of maintenance and operation.

"In dams of magnitude, or where the engineering features involved are such as to require it, or when controversial issues are involved, the State Engineer avails himself of the services and advice of consultants experienced in the particular phase under consideration to report upon these technical matters that a proper and sound solution of the problem may be reached.

"The personnel and activities of the Department must of necessity permit of extreme flexibility to meet a wide variety of activities."

The Department confines its efforts to the approval or disapproval of applications and to ensuring the proper execution of the work in accordance with the plans and specifications. This important policy was further described by Mr. Hawley, as follows:

"It is the aim and endeavor of the Department, rigidly adhered to, to require that personnel refrain from forming conclusions on the basis of local or political influence, imposing unwarranted or dictatorial conditions beyond the requirements of safety, exerting unnecessary influence over construction, assuming of engineering direction, and to avoid economic considerations,

such as choice of type, location, etc. It is likewise the desire of the Department to minimize inconvenience to the owner, eliminate transgression on the Engineering Profession, and cause a minimum disturbance of economic conditions. On the other hand the personnel stands at all times willing to discuss informally with the engineer acting for the applicant controversial problems of design or construction relating to the safety of the dam."

In closing, Mr. Hawley endorsed the principle of State supervision of dams, as follows:

"State supervision of dams, if properly and competently administered, and if directed aggressively to the proper requirements of safety in dam design and construction, merits the support of the Engineering Profession for the arousing of public confidence and the technical advancement that will accrue to the profession; assures the public of unbiased and uninfluenced engineering opinion; dispels inherent public fear of dams; centralizes, and makes uniform, co-ordinated control; and makes available in condensed form recorded technical information and data of inestimable value. State supervision of dams should, however, be ever cognizant of the fact that the advancement of any community depends upon the development of its water resources through the construction of dams. This program must not be retarded through overcautious and unwarranted functioning of the office having jurisdiction beyond the requirements for assurance of safety."

GENERAL DISCUSSION

EXTREMES IN THE POLICY OF STATE SUPERVISION

By D. C. Henny,* M. Am. Soc. C. E.

Recommending to Mr. Hinderlider the need of presenting the full details concerning the dam failures presented in the paper as read, Mr. Henny declared:

"I do not question at all the accuracy of these statistics. I merely question the correctness of the impression they leave with the average reader. * * * I am now under the impression that a great majority of dams contained in that 4% are minor cases, small dams, possibly built by farmers, possibly built without any engineering supervision whatever, and their destruction is doing but a minor amount of damage."

One gathers the impression from the paper that the author recommends giving such authority and jurisdiction to the State that it might be burdensome on the builder, financially and otherwise. Then, said Mr. Henny, after assuming the entire authority, the State leaves the owner merely to follow instructions or abandon the project. Quoting Mr. Henny directly:

"If we approach this subject in a sensible way we see there the two extremes, complete authority on the one hand and no liability on the other. To my mind that simply means this: That whatever laws our various States pass for the protection of the public, it is a question of the application of that law in a common sense manner whereby the officer in authority shall exercise that authority in a sensible, practical, engineering way."

* Cons. Engr., Portland, Ore.

Under State supervision, the mistake of giving a single man authority to design and construct a dam should be eliminated, according to Mr. Henny. Another feature that should be stressed, he thought, was that of inspection. Finally, in commenting on the drafting of the California law, Mr. Henny stated that engineers in the Western States had watched its development with interest and were now watching the effect of its operation. In the drafting of that law, 95% of the credit should go to California engineers, in the opinion of Mr. Henny.

PUBLIC VS. PRIVATE OWNERSHIP

By C. E. Grunsky,* Past-President, Am. Soc. C. E.

States that have allowed private corporations, or restricted districts or municipalities to construct dams, have been in error, in the opinion of Mr. Grunsky. To quote:

"The conservation of the most important natural resource which we have—our water—should be with the State and with the United States. Every storage of any importance should be directed, should be constructed at State or National expense, and the output should be placed at the disposal of those that can use it. It should be wholesale, not retail.

"That does not mean that when a comprehensive plan is made we will not permit an individual, a corporation, or a municipality, to make the development; but the State should stand back of it; it should be the State plan and the State should be responsible.

"I believe in State control of the reservoir; I believe in National control of the reservoir when a river is an interstate or international river. I think we have made a mistake when we have given away, or allowed to be acquired by private interests, our reservoir sites. They should have remained in public control."

STATE SUPERVISION A PUBLIC SAFETY MEASURE

By A. H. Markwart,† M. Am. Soc. C. E.

In commending Mr. Hinderlider's paper as a valuable contribution, Mr. Markwart deplored the enactment of too much legislation. However, he said,

"The building of dams is done by many people, not by one solid group, and consequently it is difficult to control from within. Perhaps this is one of the subjects in connection with which we must have legislation and perhaps that legislation is best handled by the States. * * * I think our legislation must run along the lines that it is a public safety measure, a preventative measure, you might call it, like many other things that are provided in the Health Service. If they provide against a typhoid fever epidemic they do not say we are going to have one; but the authorities will provide measures that will take care of it in case it comes, or perhaps prevent it from coming."

* Cons. Engr. (C. E. Grunsky Co.), San Francisco, Calif.

† Vice-Pres., Pacific Gas & Elec. Co., San Francisco, Calif.

Such legislation should not be construed to relieve the owner or his engineer from responsibility in connection with the construction of dams, because, to quote Mr. Markwart, the State Engineer "will need all the help he can get, and it is up to all of us who have that kind of work to do, to co-operate in every way."

THE MENACE OF PREJUDICE

By Richard R. Lyman,* M. Am. Soc. C. E.

A hypothetical case of a city governed by one political party, attempting to build a dam that would be subject to supervision by the State, which might be governed by an opposing political party, was described by Mr. Lyman. In such a case, he said,

"It seems to me there ought to be somewhere an authority to which the little engineer, who has made the design, may appeal if the State Engineer is a prejudiced judge."

CLASSIFICATION OF DAM FAILURES

By J. B. Lippincott,† M. Am. Soc. C. E.

Commenting on Mr. Hinderlider's list of dam failures, only a very brief summary of which was read at the meeting, Mr. Lippincott offered the suggestion that possibly the classification was too severe.

For example, in another list cited by Mr. Hinderlider, at least three dams were listed as failures when for all practical purposes they had not failed. At the Gibraltar Dam there was a serious erosion of the spillway below the dam which was repaired without serious damages or loss of any water. The case of the Sweetwater Dam was similar. The failure of the Pudding Stone Dam consisted of erosion of a few thousand yards of earth. This occurred during construction when a flood overtaxed the capacity of a temporary by-pass tunnel.

STATE SUPERVISION IN OREGON

By F. R. Schanck,‡ M. Am. Soc. C. E.

The Portland Section of the Society played a laudable part in framing the Oregon law providing that the State Engineer shall have supervision over all kinds of hydraulic structures. Mr. Schanck declared that the Section

* Civ. and Cons. Engr. (Lyman & Pack), Salt Lake City, Utah.
† Cons. Hydr. Engr., Los Angeles, Calif.
‡ Cons. Engr., Portland, Ore.

found the members of the Legislature very glad to embody in the law practically all ideas suggested.

Describing the law very briefly, Mr. Schanck stated:

"The State Engineer, who possibly may not be especially qualified to pass on structures that might come before him, may employ a board of consulting engineers, or geologists; and one of the provisions that we arranged to have included in that law was that the State Engineer must visit the site. I think we will all recognize, and especially from the statistics given by Mr. Hinderlider, that the conditions upon which the dam is founded are usually much more important than the academic design, as any good engineer, after he has been graduated and studied stresses, can generally make a design that will work if he knows upon what it is going to be built. And then there is a review by the Court of any difference that may develop between the State Engineer, and his findings, and the owner. It is a very brief law."

HIGHWAY DIVISION

APRIL 24, 1930

Morning Session—9:10 A. M. to 11:50 A. M.

PRE-QUALIFICATION OF CONTRACTORS FROM THE STANDPOINT OF THE ENGINEER

By C. H. Purcell,* Assoc. M. Am. Soc. C. E.

In this paper, Mr. Purcell reviewed broadly the arguments for and against pre-qualification of contractors as presented in recent years. The paper also contained a brief résumé of California practice in this respect.

Responsibility of Contractors

For a test of responsibility, Mr. Purcell cited the standard recommended by the Committee on Ethics of the Associated General Contractors of America. This Committee recommends that the contractor should have:

(1) Previously executed a similar contract of at least one-half the amount of his bid;
(2) Ample and suitable equipment definitely available;
(3) A clean record of past contracts successfully fulfilled;
(4) A generally high reputation founded on consistent performance and good character; and
(5) A net worth of sound and liquid assets equal to 20% of all the work under way; including the new bid.

Commenting on these requirements, Mr. Purcell stated:

"Responsibility is elastic. The size may have no bearing on the responsibility of a contractor; the small firm may be responsible within the class of work which he has followed and within the size of project he has in the past completed. The large contractor may not be responsible outside the class of work which he has heretofore been performing or outside the size of contract which he has heretofore performed. Either one, within his class of work, may over-extend his operations, take on too many contracts, become overloaded and thereby become irresponsible. The contractor who unbalances his bid, gambles on conditions rather than make a careful study, pursues unethical methods or furnishes inferior materials, thereby producing an inferior product, is irresponsible. Irresponsible has a real meaning only when it is applied to a specific project."

Within the limits of this definition, according to Mr. Purcell, a responsible contractor will make his bid according to an intelligent appraisal of estimated costs. The irresponsible contractor may have no organization or he may have no basic standard upon which to base estimates. In short,

* Chf. of Div., and State Highway Engr., Div. of Highways, Sacramento, Calif.

"The responsible firm that has made an honest bid based on true costs has no chance against the irresponsible one that does not know what the work is worth and that is not equipped to carry out the work. The belief that price alone is to be considered because inspection and bond will insure quality of work, is erroneous."

The Lowest Responsible Bidder

The awarding officer is remiss, if he awards contracts to the lowest bidder without inquiring as to responsibility. Mr. Purcell declared that Courts will uphold such an officer who rejects the very lowest bid for cause. However, other factors, such as the interested surety, a bank, or some political influence, are sometimes brought to bear. The result is, said Mr. Purcell,

"That often an entirely inaccurate and erroneous impression is given the public to the discredit of the awarding official. Such a situation requires an awarding official of more than average integrity, courage, and judgment in order to withstand the criticism to which he may be subjected and secure a responsible contractor."

The Public Pays

That a contractor has been able to secure a surety bond is not sufficient pre-qualification because, as pointed out by Mr. Purcell, the surety company is interested primarily in the contractor's ability to pay a loss that may be incurred in the performance of the contract. Quoting from the paper,

"A surety bond does not protect against loss of the use of the project owing to the contract not being completed; neither does it cover the wear and tear on vehicles which are forced to use a detour for a longer period than should have been necessary; nor does it cover repair and replacement cost due to improper construction which may not be discovered until a later date. The question may be asked as to who does pay all this expense which may be either directly or indirectly a result of an irresponsible contractor. The answer is that the public pays."

Methods of Qualifying as a Contractor

Two general methods of procedure are in common use. One may be termed post-qualification and the other pre-qualification. The distinction made by Mr. Purcell was that in post-qualification bids were received more or less indiscriminately and the lowest most responsible bidder was selected. By pre-qualification, he said, bids were received only from responsible bidders and the contract was awarded automatically to the lowest of these.

Six objections to pre-qualification may be listed, namely:

(1) Questionnaires containing the financial statement and experience record are too complicated and expensive to fill out;

(2) By such pre-qualification, the contractor's private affairs may be exposed to public scrutiny, thus giving competitors an unfair advantage;

(3) It restricts competition and gives too much power to the awarding official;

(4) Small firms are thus prevented from bidding;

(5) There is no appeal from the decision of the awarding official; and

(6) A certified check should be considered as adequate pre-qualification.

To these, Mr. Purcell offered the following answers:

(1) As a business man, even a small contractor should have knowledge of his financial standing. Furthermore, the Income Tax Laws require it of him.

(2) Financial statements and this information, submitted for the purpose of qualifying, are confidential and should be so treated.

(3) The awarding official has no more authority under pre-qualification than under post-qualification. He merely exercises discretion before, rather than after, the bids are opened.

(4) Responsible small firms will not be barred unless the project is so much larger than their acknowledged scope that they are thus disqualified.

(5) Most organizations that require pre-qualifications have a board of appeal where a disqualified contractor may review his case.

(6) The ease with which certified checks may be secured makes it plain that this feature has no bearing on the competing of the bidder.

On the other hand, to quote Mr. Purcell directly,

"The advocates of pre-qualification claim the following to be among the advantages of such a system:

"The incompetent or irresponsible contractor is advised of that fact before he has gone to any expense in estimating a project. With the other method, that is post-qualification, the contractor may proceed to incur expense of figuring the job not knowing that when he does submit his bid it will be rejected on the ground that he is not qualified to handle that particular project. He is therefore saved money by pre-qualification and is also relieved from the embarrassment of having his bid thrown out after bids are opened. Since the bidders have all been investigated and their qualifications reviewed with reference to the project on which they are bidding, all bids received are from contractors who have been judged competent and responsible to handle the particular job. All that remains is to determine the lowest bidder. The temptation of the awarding official to follow the line of least resistance and award to the low bidder, whether he is competent or not, is removed because competency was determined before the bids were submitted."

Maximum Requirements

As yet, according to Mr. Purcell, no hard-and-fast rules have been established as to requirements of pre-qualification. He offered these items, namely, (*a*) the contractor's skill or ability; (*b*) his reputation for honesty; and (*c*) his financial ability. Commenting on these,

"Obviously, there is no mathematical formula for computing a bidder's skill, but a reasonably accurate idea can be obtained from his record of previous construction projects successfully completed. The bidder's reputation for honesty is especially important in reference to the quality of work performed. No amount of supervision and inspection can force a first-class job out of a contractor who is unwilling to produce such a job or who is incompetent to do such work. Honesty, like skill, cannot be accurately measured but a sound opinion can be formed by reviewing the record of his previous contracts. Financial ability can be quite accurately gauged. Measurement of a business man's responsibility by means of his financial statement dates almost from time immemorial. This factor is quite often overlooked in reviewing a firm's qualifications."

Several considerations should govern a contractor in bidding on work. For example, he should not attempt to accept work more than two or three

times the estimated cost of the largest previous project of similar character completed. The working capital, or net liquid assets available for the project, should also be taken into account. The amount necessary will depend upon the character of job, according to Mr. Purcell. On highway work, for example, the contractor should have equipment paid for and working capital equal to 10% of the project price. Mr. Purcell gave his opinion that if equipment was inadequate, the available working capital should be 15 or 20% of the total cost of the work.

Pre-Qualification in California

As the result of a law passed by the Legislature in 1929, the California Department of Public Works is authorized to refuse to furnish plans and specifications to, or to receive bids from, contractors whom the Department believes to be unqualified, according to Mr. Purcell. Another act, bearing on the same subject, was passed in 1929; it requires all contractors operating in California to be licensed.

Two divisions of the Department of Public Works—Highways and Architecture—have issued questionnaires to contractors as a part of their pre-qualification program. Mr. Purcell described the form of this questionnaire as substantially that recommended by the Joint Conference on Construction Practices, Clearing House Section of the American Bankers Association and the Associated General Contractors of America. Mr. Purcell's account of the methods of correlating data is as follows:

"In the Division of Highways the Statements of Experience and Financial Condition are reviewed by a board designated by the State Highway Engineer, and then they are classified as to the class of work the contractors are qualified to handle. In reviewing the questionnaire a thorough audit of the financial statement is made by the Chief Accountant. The classes of work tentatively employed are grading, paving, surfacing, and bridges. It is not attempted to fix a definite maximum limit on contractors at the time their statement is reviewed. However, a tentative maximum limit is fixed which may not be exceeded without a special review of the contractor's statement by the Board. Doubtful cases are referred to the State Highway Engineer. The contractor's classification and qualification are based on:

"1.—The kind of experience he has had.

"2.—The kind and condition of his major equipment, including information as to the purchase price, age, depreciation, etc.

"3.—His record on previous work as it indicates efficiency in prosecuting the work, knowledge of how to do that kind of work, attitude toward the work and toward the State—that is, whether he is willing to produce a first-class job, or attempts to rob the job, and whether he is on the lookout at all times for 'extras.'

"4.—Financial condition as indicated by his statement (which is carefully reviewed) and a short statement made showing net worth and net liquid assets."

The contents of the statement are considered strictly confidential despite the fact that there have been requests for this information from surety companies, credit associations, and others.

Quoting Mr. Purcell,

"In order that the Department may have reliable information at hand on which to judge the contractor's fitness for future work, and in order that the Department may know whether or not it is justified in increasing the maximum limit of capacity of a contractor, a complete file is maintained on each contractor. This file contains information on the following matters:

"(1) Pre-qualification questionnaire submitted by contractor and on which he is qualified.

"(2) A statement of current contracts, both with the State and with other organizations, in so far as this record can be kept.

"(3) A performance record is kept which will have a bearing on the contractor's eligibility and capacity for future work. This record contains the following data: (*a*) Attitude toward work: whether or not the contractor attempts to do a first-class job or attempts to skin the job; (*b*) attitude toward the State: whether the contractor is willing to co-operate with the State or takes an antagonistic attitude; (*c*) prosecution of work: whether the work is undertaken and carried out with due diligence or whether there is unnecessary delay; records of overrun of time are kept in connection with this; (*d*) knowledge of work: whether or not the contractor knows how to handle that particular kind of work; (*e*) equipment: whether the contractor's equipment is suitable for that kind of work, also whether it is in good condition or not; (*f*) quality of work; (*g*) claims for 'extras': whether or not the contractor makes a practice of claiming 'extras' on every little detail and is always on the lookout to make a claim on the slightest excuse."

A Public Benefit

Organizations that have adopted the policy of pre-qualification are strongly in favor of it, according to Mr. Purcell. In California, the Associated General Contractors of America and the various surety companies co-operate and are anxious to see the plan given a fair trial. In closing, he expressed the hope that pre-qualification will improve business relations between the State and the contractor, and that, if properly administered, it will result in the public "securing a uniformly high quality of service without increased cost."

PRE-QUALIFICATION OF CONTRACTORS: LEGAL ASPECTS

By L. I. Hewes,* M. Am. Soc. C. E.

Introducing his subject, Mr. Hewes explained the attitude of the U. S. Bureau of Public Roads with respect to pre-qualification, as follows:

"Actual pre-qualification of bidders (or of prospective bidders) establishes a group of qualified bidders in advance of the opening of bids. It is a practice begun by the Bureau of Public Roads tentatively in 1923, and universally followed in the Western lettings in 1928 and 1929. The Bureau procedure is based on interpretation of the administrative authority of the contracting officer. It has been productive of good results.

* * * * *

"Authority to withhold the bidder's sheet from a contractor to whom award would not be recommended has never been questioned. Authority to

* Deputy Chf. Engr., U. S. Bureau of Public Roads, San Francisco, Calif.

require a qualification statement in the form of the Joint Conference Questionnaire has also never been questioned. These are the two essential elements of pre-qualification of bidders.

"The Bureau has never adopted arbitrary or fixed stipulations. It has never refused personal interviews between prospective bidders and contracting officers (or their representatives). It has never made an issue of marginal cases. It has always given prospective bidders the benefit of the doubt, but has seldom been embarrassed by weak bidders."

In other ways, Mr. Hewes pointed out, the practice had found favor. For example, in the Western forests and parks, it has put highway contracting on a sounder basis. It has also given confidence to equipment manufacturers, bankers, and surety people. It has helped some contractors to avoid disaster.

The principle of requiring contractors to establish their qualifications before submitting bids is enforced, either by legislative enactment or by administrative procedure, in California, South Carolina, Georgia, Missouri, Tennessee, Wisconsin, Iowa, Kentucky, South Dakota, and Texas. In view of this general acceptance, Mr. Hewes propounded the question, "What, if anything, is against it?" The paper was concerned with discussing this question from a legal point of view, especially as applied to three adverse developments, namely, the failure of proposed legislation in Oregon in 1929; the act of the Governor of Pennsylvania in 1929 when he vetoed the pre-qualification legislation placed before him; and in the City of Philadelphia, the case of a bidder who secured a favorable decision from a lower Court to restrain the City from enforcing its pre-qualification ordinance.

Oregon's Experience

In Oregon, according to Mr. Hewes, the proposed bill (House Bill 429) provided that for contracts exceeding $25 000, the awarding officers should require, before delivering plans,

"A sworn statement of financial and general ability, equipment, experience in construction of public improvements, and of such other matters as such public officer may require for determination for the benefit of the public in the performance of any such contemplated public improvements; and such statement with any subsequent amendment thereof shall be in writing on a standard form of questionnaire to be furnished by such public officer, and shall be filed with public officer prior to the date upon which bids for the performance of public contract are to be opened, which statement shall not be disclosed except upon written order of such person or persons or on subpœna in accordance with law."

As explained by Mr. Hewes, the qualifying officer was allowed a maximum of seven days to pass judgment on this statement and if the applicant was rejected, a written statement containing reasons was stipulated. On appeal within five days thereafter, the Circuit Judge of the county was empowered by the Act to modify, reverse, or set aside the decision of the contracting officer if such action occurred within ten days after the date of appeal.

For the remainder, quoting Mr. Hewes:

"Perhaps the most unworkable feature of this proposed Oregon bill was the following:

"'The letting of any contract for a public improvement concerning which a prospective bidder has taken an appeal shall be postponed, and no bids shall be opened or made public until such appeal has been determined!'

"Under such language a dissatisfied bidder could, under unfavorable conditions, hold up the award of important contracts about three weeks.

"The Oregon bill also provided that the contracting officer should announce the approved prospective bidders before opening bids. The contracting officer might also require a bidder to withdraw to prevent his becoming over-extended.

"While this Oregon act was pending, over-zealous partisans sponsored the submission of a bogus bid to the Highway Commission, and this hoax undoubtedly helped defeat the bill. Probably also the mandatory character of prequalification of bidders for all County and municipal contracts caused opposition."

Describing the instance in which the Governor of Pennsylvania vetoed a proposed bill, Mr. Hewes quoted the Governor's statement issued in connection therewith, as follows:

"'It places unlimited discretion in one city official to disqualify bidders without giving him opportunity for redress if the one's decision was arbitrary, unreasonable, or unjust.'"

THE PHILADELPHIA CASE

As a prerequisite for understanding this situation, Mr. Hewes explained that,

"On June 14, 1929, the Council passed an ordinance providing for the pre-qualification of bidders on city work. This order provided that the prospective bidder should file evidence that he 'has the necessary facilities, experience and financial resources to perform the work in a proper and satisfactory manner within the time stipulated,' and a statement as to plant facilities, amount of other contracts, etc. In the case of refusal to file a statement, or in a finding that the prospective bidder was unqualified, the contracting officer was authorized to disregard his bid. In the words of the ordinance, a dissatisfied bidder

"'* * * may, within 24 hours after receipt of such notice, request a hearing before a board to be composed of the said awarding officer, and two other heads of departments, chiefs of bureau, or other city officials conversant with construction work and to be designated by the Mayor * * *'"

Late in 1929, according to Mr. Hewes, the Court, ruling in favor of a plaintiff against the City, decided that the ordinance was illegal; but subsequently this ruling was set aside by a higher Court under the agreement that "the ordinance of 1929 is fully authorized by the permissive language of the Act of 1874 and fully complies therewith."

Still later, the Supreme Court of Pennsylvania reversed the second decision and upheld the position of the Appellant. In its decision, the Court declared that the ordinance in question was unobjectionable in respect to preliminary determination of a contractor's responsibility; but it questioned whether the method ordained actually had the sanction of the statute.

Finally, as described by Mr. Hewes, the Court stated its opinion that (1) all bidders on municipal contracts must be treated the same; (2) that the City may accept and schedule all bids and then, acting in good faith, may refuse to award the contract to the lowest bidder because he is not the lowest responsible bidder; or (3) the City may determine the responsibility of bidders

in advance and refuse to receive bids from those who, after treating all alike, are found to be not responsible. However, to quote the Court, the City,

"'May not impose conditions on one prospective bidder, which are not imposed upon all; nor enforce a method by which, through favoritism, one person may be conclusively authorized to bid on a pending contract, while another, equally as responsible and perhaps more so, is wholly excluded from even submitting a bid.'"

Lessons Derived from Adverse Experience

Summarizing the various phases of the foregoing three cases, Mr. Hewes declared that they contained not much argument against the pre-qualification principle. Quoting:

"Where new legislation is required, the evidence indicates that it is preferable to have: (1) Sufficient time for contractors to present their qualifications in advance; (2) to have those qualifications passed upon by a board or committee, rather than by an individual; (3) to avoid any variation whatever in the treatment of prospective bidders; and (4) to avoid any method of appeal that may delay the letting. It may be questioned whether at this time it is advisable policy for legislation to attempt to cover the entire field of public contracts; that is to say, to make it mandatory for all cities, towns, and counties to so proceed. The Supreme Court of Pennsylvania explicitly sanctioned the principle, and questioned only the method of pre-qualification. The Governor of Pennsylvania questioned one-man authority. The failure in Oregon was evidently due to clumsy provisions."

The California Law

In the opinion of Mr. Hewes, the California law of 1929 contains the essence of good practice for pre-qualifying bidders. For example, Chapter 644 of that law states:

"The Department of Public Works may, within its discretion, before furnishing any person proposing to bid on duly advertised public work, with plans and specifications for the proposed public work, require from any such person answers to questions contained in a standard form of questionnaire and financial statement, including a complete statement of the person's financial ability and experience in performing public work.

"Whenever the Department of Public Works is not satisfied with the sufficiency of the answers contained in such questionnaire and financial statement it may refuse to furnish such person with plans and specifications on any such duly advertised public work, and the bid of any person to whom plans and specifications have not been issued must be disregarded."

PRE-QUALIFICATION OF CONTRACTORS FROM THE STANDPOINT OF THE CONTRACTOR

By Walter Wilkinson,* Esq.

"Responsible contractors are looking anxiously forward to the arrival of the day when the American public will fully recognize distinction between classes of contractors, and will cease their past practice which has continually introduced irresponsible and dishonest concerns into this most important

* Pres., California Branch, Associated General Contractors of America, Watsonville, Calif.

industry and maintained them there at the expense of the reputable contractors and all other allied interests."

In this terse statement, Mr. Wilkinson expressed the consensus of opinion from the standpoint of the contractor. They have come to realize, he said, that,

"The cheap-price method in construction encourages non-profit operation, bringing about the worst and most serious result of all, which is cheap and poor construction. The non-profit operation deprives the Government of taxes, while the cheap and poor construction creates untold expense and endangers life. In the last analysis under the cheap-price method, neither the owner nor the public are profiting because they are getting only what they pay for and no more. Any public official, or any owner, who will try to get a contractor to take work for cost, or less than cost, and any surety company that will underwrite a contractor taking work on that basis, is certainly inviting disaster."

What Is a Responsible Contractor?

In the opinion of Mr. Wilkinson, the term, "responsible contractor," designates one who enters into his agreement expecting to abide by the plans and specifications. A contractor may be responsible in different degrees, depending on the class of work to be done. For example, said Mr. Wilkinson,

"The responsible small contractor is often a man, or a concern, who may not desire to do a great volume of business. He has a reputation for honesty and integrity in his community and remains responsible by virtue of having brains enough not to 'bite off more than he can chew'."

Difficulties in Rejecting a Low Bid

It is a deplorable fact, he observed, that only two qualifications have ordinarily been demanded of a bidder, namely, that he submit the lowest proposal and that he provide a surety bond. Commenting further on this state of affairs, Mr. Wilkinson said,

"Ability to manage construction, technical skill in the particular type of work, sufficient equipment, financial resources, and, many times, even honesty have been regarded as entirely superfluous. The public official has, commonly, been confronted with a discouraging task when he has tried to disqualify an irresponsible bidder. Although the engineer may have known that the bidder was insolvent and absolutely incompetent to perform the work, yet he has found, when he tried to prove it that he was opposed from every turn by surety agents, material dealers, machinery distributors, politicians, and even, sometimes, bankers who have shown up to testify as to the bidder's financial worth, neglecting to tell how much money the said bidder was owing and that they each were hoping to avoid a loss.

"The awarding officials have not always been able to rely upon the sworn statements of the bidders, and have tried to check them up; yet the sources of information which should be at their disposal for that purpose, have not been in existence. There has been no particular place where the records of contractors were compiled. Often contractors who were in default on one job, while bidding on another, were, at that time, insolvent, but managed to conceal the fact."

Furthermore,

"The public has been wrongfully taught to think in terms of cheap price, when it should have been taught to think in terms of safety, durability, per-

manency, and less future maintenance costs. Almost universally his cheap price, and not his qualifications, has determined the contractor, and this has resulted in his taking the work too cheaply in order to get the job, with the hope that something would happen to bring him out with a profit. Competition in public construction has been open to the world, with no price advantage given to the well-financed, well-equipped, and properly manned constructing organization which can, will, and does, furnish a good job, built on time, and without trouble."

Pre-Qualification

One of the most important steps to be considered in alleviating this condition is pre-qualification of contractors, in the opinion of Mr. Wilkinson. Speaking for contractors in general, he said,

"We believe that the benefits to be derived from this system will be very significant, both to the bidder and to the public, for thereby we will learn our limitations before going to the trouble and expense of bidding and, some possibly, may be saved from disaster. Thereby awarding officials will have a chance to exercise discretion unhampered by any agencies that have interests at stake. These agencies will have no interest in a specific bidder before the award, and will not be on hand to thwart the action of the conscientious awarding official.

* * * * *

"Under the pre-qualification system the public official who acts as an agent for the surety company will have no opportunity to inject his hand into the situation."

Organized Contractors

Through the medium of an organized group known as the Affiliated Bureau, the co-operating and co-ordination of related institutions and industries are being brought about for the best interests of all concerned. Mr. Wilkinson declared that this branch of the Associated General Contractors of America has taken the initiative in showing the advantages of pre-qualification. As a result, he said, the Bureau of Contract Information, Incorporated, has been organized for the purpose of co-operating with those responsible in the award of construction contracts. Its specific functions were given by Mr. Wilkinson, as follows:

"1.—To establish definitely the facts concerning the performance record of the individual contracting concerns throughout the United States, as developed during the last three years.

"2.—To carry forward the individual performance records when such facts have once been established, and as further contracts are awarded, bonded, and performed.

"3.—To disclose verified information about the performance records of the individual contracting concerns to those legitimately entitled to it by reason of their being responsible for the award of public and private construction contracts, their writing of contract bonds, or their extension of credit."

As a part of the program the Bureau is investigating the three-year performance record and the regulation of every general contractor in the United States. In this way, according to Mr. Wilkinson,

"The records of all general contractors, whether they fill out the questionnaire or not, will be built up by checking with the numerous agencies avail-

able for that purpose. Verified information concerning any contractor will be made available to owners, and others with whom he has business dealings."

In closing, he urged the co-operation and assistance of the responsible men in the ranks of the related trades and professions—among surety men, equipment men, material men, and engineers.

GENERAL DISCUSSION

PRESENT STATUS OF PRE-QUALIFICATION IN PHILADELPHIA, PENNSYLVANIA

By C. H. Stevens,* M. Am. Soc. C. E.

As a result of adverse experience with pre-qualification public officers in Philadelphia are somewhat discouraged concerning it. Mr. Stevens pointed out that the present Act empowered the Executive Director of a City Department to investigate the qualifications of bidders and that this is a duty that they are loath to relinquish.

In the opinion of Mr. Stevens, the decision of the Supreme Court of Pennsylvania, cited by Mr. Hewes, seemed to imply that the establishment of a Reviewing Board composed of persons not connected with the Department in question would guarantee to the bidders a more reasonable chance of unbiased consideration.

The practicability of this attitude was questioned by Mr. Stevens since, he said, the engineer is peculiarly qualified to handle the phase of the work concerned with qualifications of contractors, their financial equipment, experience, etc. A separate Board as suggested might be composed of members with no knowledge of engineering or construction problems.

Incidentally, Mr. Stevens declared that the Department of Transit in Philadelphia had administered a total expenditure of about $100 000 000 in the past six years, and that each contractor had been given an opportunity to do his work under fair conditions.

LOW-COST, BITUMINOUS-TREATED, CRUSHED ROCK AND GRAVEL ROADS

By W. N. Frickstad,† M. Am. Soc. C. E.

Under the complete title, "Further Developments in the Construction of Low-Cost, Bituminous-Treated, Crushed Rock and Gravel Roads," Mr. Frickstad outlined the progress made by Western States in this phase of road construction since the latter part of 1928. For a survey of the status of the art previous to this date Mr. Frickstad gave a summary of a paper entitled "Bituminous Treatment of Fine Crushed Rock and Gravel Roads", by Thomas E.

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† City Engr., Oakland, Calif.

Stanton, Jr., M. Am. Soc. C. E., presented at the Fall Meeting of the Society at San Diego, Calif., on October 4, 1928. At that time Mr. Stanton described two types of treatment, involving fine-crushed aggregate with a light type of asphaltic binder known as "fuel oil."

The two types are abstracted by Mr. Frickstad as follows:

"(1) Surface treatment, sometimes called penetrative treatment because the binder is to a degree absorbed into the upper strata of aggregate. Construction starts with a thorough preparation of the base, which implies repairs, thorough compaction, and thorough cleaning. On the prepared base is spread approximately $\frac{1}{4}$ or $\frac{3}{10}$ gal. of asphaltic fuel oil per sq. yd., containing 60 to 70% of 80 penetration asphalt and having a specific viscosity (Engler) at 50° cent. of 10 to 25. This oil is allowed to penetrate the upper strata of the metal. Sometimes stone chips are spread to protect traffic. After a suitable interval another application of asphaltic oil is spread, which may be of the same nature as the primer or may be of slightly higher viscosity, or may be much heavier, approaching pure asphalt. Stone chips are then spread in sufficient amount to absorb the oil, and the road is rolled. Details vary considerably, particularly in the amount of oil and stone chips added, which is higher for the heavier oil. Sometimes the second application of oil is kept to a minimum, usually $\frac{1}{8}$ to $\frac{1}{4}$ gal. per sq. yd., the stone chips are relatively large, as for example $\frac{5}{8}$ to $\frac{7}{8}$ in., and a third spread of oil of approximately the same amount is applied and followed by finer stone chips. Heavy oil, applied hot or emulsified, is considered more durable than fuel oil, particularly against weather damage.

"(2) Oil mixing treatment, in which the asphalt oil binder is thoroughly mixed with the upper 2, 3, or even 4 in. of surfacing material. The first construction operation is to provide the desired amount of loose mineral aggregate on top of a firmly compacted base. The aggregate should all pass a 1-in. screen and one-third to one-half should pass a 10-mesh sieve. It may be obtained by scarifying if sufficient be already on the road, or it may be added for treatment. The oil is usually spread in three applications, aggregating 1.25 to 2 gal., or more, per sq. yd. After each application partial mixing is accomplished with a disk or spring tooth harrow, or both. After all the oil is applied, mixing is completed by casting into a windrow with a blade grader and moving the windrow back and forth over the road, turning the mass at each move. The oil commonly used contains 60 to 70% of 80 penetration asphalt. The viscosity is variously specified, generally 25 to 45 (Engler Specific) at 50° cent., or 45 to 80. The amount of oil varies according to the grading and other characteristics of the aggregate. As with asphaltic concrete, generally the more fine material the higher proportion of oil is required. The quantity of oil is sometimes estimated by a formula suggested by C. L. McKesson, Assoc. M. Am. Soc. C. E., sometimes by appearances, but quite commonly by a modification of the stain test so long used in sheet asphalt practice. After the mixing is completed, the material is carefully spread with a blade grader to uniform thickness, and is compacted by traffic. During the compacting period, ruts and other irregularities are eliminated by a drag or a blade until traffic makes no impression, and until the road rides as smoothly as the most carefully constructed high type pavement."

The formula mentioned is:

$$P = 0.015a + 0.03b + 0.17c$$

in which,

P = percentage of oil required;

a = percentage of aggregate retained on a 10-mesh sieve;

b = percentage of aggregate passing a 10-mesh sieve; and,

c = percentage of aggregate passing a 200-mesh sieve.

Development in Surface Treatment

Mr. Frickstad reported that in the eleven Western States the 3 600 miles of State highways had been oil-treated at the end of 1928. About 2 500 additional miles of road were treated during 1929, raising the total to 6 100 miles. The present trend, according to Mr. Frickstad, is definitely toward the use of heavy oil (94+ asphalt content) in the treatment of road surfaces. Quoting directly:

"In 1929 fuel oil was used principally as a primer or as a dust palliative. A few of the less traveled roads were surface treated with fuel oil alone, but generally the fuel oil application was or will be followed with heavy oil and coarse stone chips. Thus, there is a reversion to the methods by which macadam roads have been successfully treated during the past twenty years or more. Although the process has been very satisfactory on macadam, it has not heretofore been commonly successful on gravel or fine crushed rock. The introduction of a primer seems to have overcome the difficulty.

* * * * *

"Details of surface treatment are far from standardized. The California State Highway practice is among the more elaborate, and may be outlined as follows:

"1.—Clean, repair, and thoroughly compact the base.

"2.—Spread $\frac{1}{3}$ gal. per sq. yd. of 50-60 asphaltic fuel oil. (Specific viscosity (Engler), 10-25 at 50° cent.)

"3.—Allow the first application to penetrate. Add a small quantity of fine stone chips, if necessary, to protect traffic and to absorb oil that may settle in depressions.

"4.—Spread $\frac{1}{4}$ gal. per sq. yd. of 94+ asphaltic oil at a temperature of approximately 350 degrees.

"5.—Spread sufficient $\frac{1}{2}$ to $\frac{3}{4}$-in. stone chips to cover the oil; 120 tons per mile of 18-ft. road is the estimated quantity, but this includes considerable waste due to the effort to save labor.

"6.—Roll.

"7.—Spread $\frac{1}{4}$ gal. of 94+ asphaltic oil, as before.

"8.—Cover with $\frac{1}{8}$ to $\frac{1}{2}$-in. stone chips; 80 tons per mile is estimated, which includes waste, as before.

"9.—Roll."

Several of the States, and especially California, are firmly committed to the use of plant-mixed oil in preference to road mixing. Describing trends in the use of road-mixing machinery, Mr. Frickstad reported that,

"Most of the States are equipped with tractors and blades for snow fighting and ordinary maintenance, and have simply transferred this equipment to oiling operations during the appropriate season. Use for these other purposes has influenced the purchase of equipment for oil mixing. One State, however, has equipped itself primarily for oil mixing, and adopted the tractor or dual drive type of self-propelled blade. A group of four of these is extremely efficient. The best work is done with blades of 10 or 12 ft. in length. Regardless of the type of machinery, workmanship has steadily improved. The first thought was to regard mixing as a rather haphazard process, and too often it was such in fact. During the past year neatness and system have been notable. Partly oiled material is picked cleanly from the base and turned over and over in accurately lined windrows; after the mixing is nearly completed, a small amount is allowed to remain on the base and the final mixing takes place without the incorporation of uncoated particles. It is then spread quite accurately to uniform thickness. Even where grading of the aggregate varies from

station to station, with a consequent necessity of adjusting the amount of aggregate drawn into the mixture, the windrows become remarkably uniform in color before spreading when properly manipulated. Observing the work of a skillful crew, road mixing can no longer be regarded as haphazard."

Grading of Aggregate

Aggregate grading has not progressed toward standardization, observed Mr. Frickstad, and this confirmed his opinion that a wide range is acceptable for oil mixing. There is good reason, he said,

"To believe that gradings corresponding not only to asphaltic concrete, but to Topeka mixture and sheet asphalt, may be successfully mixed with fuel oil.

"The proportion passing the 200-mesh sieve is an important element. This has been as low as 3 or 4%, and higher than 20 per cent. Apparently, the 200-mesh material should fill the spaces between the larger particles sufficiently to reduce the size of voids to the minimum and increase their number to the maximum."

Oil

Asphaltic oil from the Rocky Mountains is being used successfully, according to Mr. Frickstad, although its characteristics are somewhat different from the California product. Commenting on the effect of such differences, he declared:

"So many factors affect the success or failure of mixtures that the influence of the oil is difficult to segregate. Plant mixtures offer a better opportunity to study the effect of oil than road mixtures. Undoubtedly oils differ in their suitability and a strong effort should be undertaken to ascertain the reason.

"After the first season of oil mixing the medium grade of fuel oil with an Engler specific viscosity of 25 to 45 at 50° Fahr., carrying 60 to 70% of 80 penetration asphalt, was generally recommended. During the 1929 season there was a marked tendency to raise the viscosity to 45 to 80, which is the grade generally used for plant mixing. The higher viscosity generally carries with it higher specific gravity, and perhaps higher ductility, but adds somewhat to the difficulty of mixing, particularly in cool weather."

Proportion of Oil

As to the proportion of oil to be used, Mr. Frickstad stated that this question is much debated:

"Possibly the disagreement is more apparent than real, for the reason that aggregates vary widely in grading and in their absorptive qualities. For estimating purposes it is common to allow ½ gal. per sq. yd. per in. of finished thickness, with 15 to 30% allowance for extra requirements. The amount used generally falls within the range of the estimate. One group of engineers advocates the least amount that will coat the particles; another the largest amount that can be incorporated without producing actual instability. The first group seemingly reasons that the surface area of particles is the significant factor, implying that the binding action of oil comes from surface tension, which in turn implies that the quantity of oil should not exceed that required to produce a film around each particle. The other group argues that the voids are most significant and must be filled with oil. In the meantime errors in the quantity of oil applied during the first treatment are so readily corrected that the difference of opinion is not serious. However, it

is obvious that the less oil used, the cheaper will be the construction cost—a statement which applies with particular force to projects distant from the source of oil supply."

Roadway sections are generally being built with a minimum width of 20 ft. and a compacted thickness of 3 in. Furthermore, Mr. Frickstad reported that costs vary widely from about $1 200 per mile to as much as $2 000 per mile. Any single project may vary even more, since the cost of the oil is the largest variant. He mentioned the fact that the cost of applying the oil was rather uniformly $500 per mile of 18-ft. road surface.

Failures

Mr. Frickstad's statement concerning failures may be quoted as follows:

"It is safe to say that the principal failures continue to arise from faulty support. Support may be deficient in thickness of metal or stability of sub-grade. As frequently said before, an impervious oil surface aggravates the effect of moisture in the sub-grade. Many oil projects being undertaken upon metal insufficient for untreated conditions, it may be expected they will be found deficient when oil treated regardless of the method used. Emulsification of the light fuel oils in the presence of moisture and traffic is frequently reported. If the moisture comes from rainfall, the mixture can be protected by application of a seal coat of heavy asphaltic oil and stone chips. Investigation shows that most cases are accompanied by a third condition, namely, the presence of an excessive amount of 200-mesh material, usually clay. Possibly the trouble may be avoided by eliminating clay from the mixture which, however, is not always practical.

"Rain during construction is a source of much annoyance. The immediate effect of free moisture is very similar to that of excess of oil, causing the material to gather in lumps while being mixed. If the moisture is retained after spreading, it produces waving and bleeding.

"The effect of moisture on the mixture during construction and afterward makes it advisable at the present stage of knowledge of the subject to proceed cautiously in moist climates. This statement is particularly true of plant mixtures, as a wet pit cannot be utilized without drying."

Emulsified and Cut-Back Binder

During 1929, said Mr. Frickstad, several projects were constructed with a road-mixed emulsified asphalt binder. The two steps in the process were:

"First, water was spread upon the aggregate prior to and during the mixing to prevent the emulsion from breaking until mixed; second, a preliminary application of ¼ to ½ gal. per sq. yd. of fuel oil was incorporated into the dry aggregate, which improved the mixing qualities of the emulsion. A seal coat of about ¼ gal. of emulsion per sq. yd., covered with fine stone chips, was found advantageous to complete either method."

Mr. Frickstad reported further that satisfactory experiments had been made using cut-back asphalt or heavy road oil as a binder for a mixing treatment. In his own words:

"In using emulsions or cut-backs in road mixtures, it is hoped, of course, to produce a harder and more stable surface than can be attained with fuel oil. Engineers and highway authorities are not quite prepared to believe that an asphalt commonly designated 'fuel oil' has sufficient binding properties to make a really successful pavement. Oil mixtures admittedly are soft and easily disturbed when first laid; asphaltic concrete hardens at once; hence it is reasoned

that the early hardening of asphalt applied as an emulsion or a cut-back is highly desirable. The truth or error of this reasoning will not be known until a number of projects have been subjected to several years of service, after which a study of their construction and maintenance costs should furnish the answer."

A test road in the Truckee River Canyon, constructed in 1929, is expected to supply much information along this line.

New Construction

In closing his paper, Mr. Frickstad offered the following survey of the construction as influenced by oil treatment processes:

"First undertaken as a maintenance measure, oil treatments of both types are beginning to have a profound effect upon design and new construction in the Western States. Few Western State highways carrying as many as 500 vehicles per day of summer traffic are being built without a definite program for oiling, and in many States the traffic limit is lower.

"Approaching the use of oil from the construction side, surface treatment has the disadvantage of requiring a well compacted base, which cannot ordinarily be had without a tedious wait or by using undesirable clay binder. Road mixing requires special equipment that contractors do not ordinarily have, and surfacing is quite commonly finished during the season when oiling is impractical. Consequently, many States are reserving oil mixing to their own forces, even if on new construction, but are adjusting the grading requirements of new surfacing material to the needs of oiling. Plant mixing has the advantage of being suitable for contract operations, which perhaps is one reason for its general adoption upon new projects in California.

"The introduction of oiling has brought about a new examination of specifications for low and intermediate types of surfacing. Much material formerly considered too fine becomes available. Binder, or filler, is being scrutinized more carefully. On the other hand, the possibilities of penetration macadam and the elimination of fine crushing are attracting attention. So far, the facility with which fine crushed material may be manipulated with machinery as contrasted with hand labor and other expensive operations for macadam shows the oil mixture to be much the lower in price, although estimates from Oregon indicate some doubt.

"Oil treatments have never been seriously proposed as a substitute for standard high type pavements, but are actually encroaching upon the field of the latter to some degree where dustlessness and convenience to traffic dominated the selection of type. Also, for lack of a satisfactory or cheap intermediate type, roads carrying 500 to 1 000 vehicles per day have frequently been paved, but oil treatment is now being found sufficient for at least immediate purposes. Thus oil treatment is finding its place, mainly at the expense of lower types, but to some degree as a substitute for higher."

DISCUSSION

MAINTENANCE COSTS OF BITUMINOUS-TREATED, CRUSHED ROCK ROADS

By E. Q. Sullivan,* Assoc. M. Am. Soc. C. E.

As a subject for discussion, Mr. Sullivan chose the 44.29 miles of State highway between Victorville and Daggett, in San Bernardino County, Cali-

* Div. Engr., California Highway Comm., San Bernardino, Calif.

fornia, which was constructed by the oil-mixing treatment. This road had a 4-in. crushed gravel surface, which was not oiled until it had been under traffic for one year. At that time, said Mr. Sullivan, it was still in fairly good condition. The cost of applying the oil-mixing treatment described by Mr. Frickstad was given as follows:

"Cost of oil furnished and spread	$577.76
Cost of mixing	498.84
Cost of oil for seal coat	94.51
Labor, etc., for seal coat	60.95
Total cost per mile for oil treating the completed road	$1 232.05"

Traffic on the road has been increasing, the present average being 866 vehicles for the 16-hour period between 6:00 A. M. and 10:00 P. M. In view of this increasing load, the maintenance costs are of interest as shown in the following tabulation:

Year.	Road surface cost per mile.	Total cost per mile.
1927–28	$ 51.54	$323.63
1928–29	92.46	174.05
1929–30	171.37	293.20

The item of $51.24 is estimated since it could not be separated accurately from the total cost in that year. Mr. Sullivan gave as the reason for the increased maintenance cost the fact that considerable attention was required on the edges of the 18-ft. surface, which was becoming too narrow for the increasing traffic. Other maintenance items, he explained, have included:

"Re-mixing a few short stretches of road surface that showed signs of being too rich in consistency with resulting corrugations. Probably not more than 1% of the total road surface has been re-mixed as a maintenance measure.

"A few broken places have developed in this road surface during the past three years, perhaps an average of one such place per mile. These broken places have been due, in general, to rodents excavating burrows under the road surface, causing failures a foot or two in diameter. The cost of such repairs is very small. The major maintenance work has been the holding of the edges of the mixed surface from breaking down."

In the opinion of Mr. Sullivan, the maintenance cost in this case could be materially reduced by widening the finished surface. On the other hand,

"Should the traffic increase on this road and more heavy trucking become established, it might be necessary to re-work the road at some future date, but it is found by experiment that this road surface can be re-mixed to a greater thickness than that now existing on the road, at small cost."

This type of surface treatment proved so well adapted for arid and semi-arid country that the work was extended into similar territory where a good, well-drained, natural sand and gravel sub-base exists.

The cost data on seven typical projects covering 133 miles of road were given by Mr. Sullivan, as follows:

ANALYSIS OF COST OF SURFACING ON SEVEN TYPICAL CONTRACT JOBS COVERING 133 MILES OF ROAD.

Location.	Miles.	Cost of Water for Sub-grade.		Cost of Surfacing in Place, Including Oil and Seal Coat.		
		Total.	Per mile.	Total.	Per mile.	Per ton.
1½ miles northeast of Yermo to 1½ miles southwest of Dunn............	20.78	$6 885.10	$331.33	$83 791.75	$3 998.04	$1.63
4 miles west of Hector to 2 miles west of Argos..............................	14.00	4 232.00	302.29	63 810.50	4 557.89	1.72
Daggett to 4 miles west of Hector....	21.22	4 361.70	205.55	101 728.14	4 793.97	1.78
Barstow to 1 mile east of Yermo......	13.05	7 040.00	539.47	60 659.50	4 648.24	1.78
1½ miles west of Siberia to 6½ miles east of Amboy..............................	22.38	17 500.00	782.30	130 437.60	5 830.92	2.19
9½ miles west of Hopkins Well to Black Butte................................	22.10	8 542.90	386.56	133 375.78	6 035.10	2.36
2 miles west of Argos to 1½ miles west of Siberia................................	19.47	16 000.00	821.78	123 045.00	6 319.72	2.38
Averages for 133 miles of construction.	18.998	$9 223.10	$481.32	$99 549.75	$5 169.12	$1.98

This tabulation differs from the previous list of data, according to Mr. Sullivan, in that it includes cost of crushing gravel and spreading the original crushed gravel surface. Mr. Sullivan explained that, under the conditions described, it has been found necessary to treat the surfaces with fuel oil and screenings. Quoting,

"In general, it is found most desirable to make the mix so dry that there is a tendency to ravel without this surface treatment. The desirable feature in requiring a dry mix lies in increased stability, resulting in elimination of any tendency of the road surface to creep or corrugate; possibility of bleeding is also eliminated.

"The surface treatment has been applied immediately after completion and compaction of the road surface. The cost of this surface treatment has averaged $155.46 per mile on day-labor work."

The factors producing the best type of work were analyzed by Mr. Sullivan as follows:

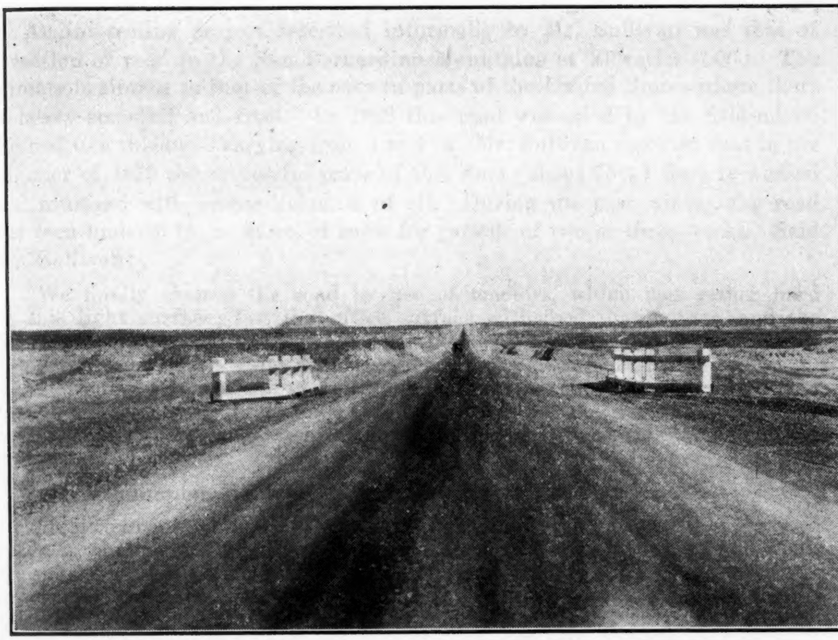
"1.—It has been found that even when the quality of materials is not of the best, reasonably satisfactory results can be obtained if additional skill and effort is put into workmanship. By this is meant, road mixing and re-mixing with careful adjustments in amount of oil.

2.—Plant mixing has brought clearly to light that of several oils passing California State specifications there is a marked difference in results.

3.—It has been found that the quality of aggregate influences results, not so much because of qualities of hardness of the particles of the aggregate as because of qualities of natural binding value in the aggregate. Clay in the aggregate, however, is found to be very detrimental. In general, an aggregate having the best mechanical lock will make the best aggregate for oil-mixing treatment.

"Some of the plant-mixed contract jobs have now been under maintenance for over one year and maintenance costs are as indicated for the road-mixed job between Victorville and Daggett, which is now three years old.

"It is believed that the plant-mixed roads can also, if desired, be re-mixed by field methods at any time, with any desired thickness of material added to increase the strength for possible increased stress of heavier traffic."



STATE HIGHWAY SURFACE IN SAN BERNARDINO COUNTY, CALIFORNIA, BETWEEN VICTORVILLE AND DAGGETT.

ANOTHER VIEW OF STATE HIGHWAY SURFACE IN SAN BERNARDINO COUNTY, CALIFORNIA.

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An interesting project described informally by Mr. Sullivan was that of a section of road in the San Bernardino Mountains at Elevation 7 000. The climate is similar to that of the eastern parts of the United States where there is heavy snowfall and frost. In 1928 this road was oiled by the field-mixed method to a thickness varying from 3 to 4 in. Mr. Sullivan reported that in the summer of 1929 the successful parts of this road (about 75%) were re-worked and re-mixed with proper amounts of oil. During the past winter, the road has been under 6 ft., or more, of snow for periods of two or three weeks. Said Mr. Sullivan:

"We finally cleared the road by use of tractors, which was rather hard on this light surface; but the entire surface withstood that winter, and the clearing operation (except perhaps 5% which will now have to be re-worked). In other words, this is quite a successful effort for decomposed granite.

"It would be impossible to build a permanent pavement at this time, even though we were ready to do that, because the fills are still very high and have a great many slides, and for a temporary measure at least this treatment has proved quite a success."

In concluding his discussion, Mr. Sullivan remarked:

"In conclusion, it is believed that the oil-mixing treatment for graveled roads is particularly suited to arid and semi-arid regions, where the roads are subjected to reasonable loads and to moderate traffic. Maintenance costs are low under these conditions. The amount of traffic that these pavements can withstand is as yet undetermined in view of there being no sign of general failure and because it is believed that the strength can be increased to meet increased future demands."

CURRENT PRACTICE IN INDIANA

By A. H. Hinkle,* M. Am. Soc. C. E.

Due to the unavoidable absence of Mr. Hinkle, his discussion of the paper by Mr. Frickstad was read by R. M. Gillis, Assoc. M. Am. Soc. C. E.

The bituminous types of surface described by Mr. Frickstad require, as a background, the use of a binder that hardens slowly so that a smooth surface can be developed during the hardening process. In the opinion of Mr. Hinkle,

"The problem so far as the bituminous binder is concerned is to select one that can be readily incorporated in the aggregate and that will give satisfactory results in holding the aggregate in place after it is incorporated. The lighter or more liquid the oils, asphalts, and tars, the better will they penetrate the fine aggregate. However, the heavier grades are more durable if they can be made to 'stick' to the surface of the aggregate which is sometimes dirty. Thus, attention is at once brought to the use and study of the emulsified and cut-back asphalts or similar acting bituminous materials, which furnish both qualities in that they are temporarily liquid with high penetration properties and after the evaporation of the water or distillate there remains the original heavy, asphaltic product that gives the necessary bond and wear. It so happens that the fuel oil of California (and some other fields

* Chf. Engr. of Maintenance, Indiana State Highway Comm., Indianapolis, Ind.

as well) are by nature, with little or no refining, in such a liquid state, and need little preparation before their use in the work.

"It is apparent that climatic and local conditions may affect somewhat the details of construction of this type of surface as well as the results secured. A more thorough understanding of the proper use of light, medium, and heavy grades of bituminous products in bituminous mixtures and bituminous surface treatments will aid appreciably in the better results secured in developing low-cost roads."

Developments in Surface Treatment.—Instead of the light fuel oil suggested by Mr. Frickstad, Mr. Hinkle mentioned the possibility of substituting a light cut-back asphalt or tar with the same viscosity, as a first surface treatment of roads covered by a fine-crushed aggregate. For the next step the use of a heavier bituminous binder will generally produce a more economical treatment. Then, said Mr. Hinkle, after a crust has been formed, the asphalt or tar to be used might be classified as medium and heavy.

These three grades, he defined in terms of relative viscosities as follows:

"The medium grade has such a viscosity that will permit dragging the treatment after the covering aggregate is applied and in the dragging operation the aggregate coated with the tar or asphalt will be deposited in the low places on the surface thus greatly smoothing it. This medium grade of tar or cut-back asphalt may vary in viscosity from 50 to 175 (Engler at 50° cent.) depending on the atmospheric temperature. Tar having a viscosity of 30 to 60 (Engler at 40° cent.), and a cut-back asphalt varying in viscosity from 75 to 175 (Engler at 50° cent.) has been frequently used for this purpose. The lighter viscosities should be used in cool weather. The heaviest viscosity that can be satisfactorily used at the available air temperatures will produce the most economical work and will be most satisfactory from the point of view of early drying. The heavy grade of bituminous material, in addition to being a heavy fuel oil, mentioned in Mr. Frickstad's paper, might be an asphalt with a penetration of 175 to 250 at 25° cent.; or a tar having a float test of 120 to 180 at 32° cent."

Beneficial Effects of Dragging.—Commenting on this factor of maintenance in passing, Mr. Hinkle declared,

"Most bituminous surfaces are subject to slight distortions and wear under traffic which gradually roughen their surface. These slight irregularities can frequently be eliminated by proper dragging with a long base drag just after surface treatments with a medium grade of tar or asphalt which treatments are applied periodically as regular maintenance operations."

Mixing Treatment.—The mixing treatment that involves light cut-back asphalt or light tar has been used extensively in some of the Central and Eastern States, according to Mr. Hinkle. Quoting from the written discussion:

"Although light and medium grades of tars and asphalt oils differing from the fuel oils used in California have been generally used in this work and although some of the details of the mixing operations have differed, the general process used has resulted in producing an intimate mixture of the bitumen and loose aggregate which is leveled and spread over the surface and compacted by traffic sometimes aided by the use of a roller. The use, in this work, of California fuel and similar oils in climates subject to heavy moisture combined with deep frost action has been experimented with enough

to indicate their possibilities. It is yet to be fully demonstrated that they will form a surface as resistant to traffic and other destructive agents as a cut-back asphalt made from an asphalt base having a penetration of 70 to 110.

"It has been observed that, where a layer of comparatively course aggregate was treated on the road by the 'mixing' process, a much more stable crust was produced. This led to the development of what is termed, for want of a better designation, a 'bituminous retread top' of 2 to 3-in. compacted thickness of about a 2-in. size stone. This differs primarily from the oil-mixing method described by Mr. Frickstad in that a coarser aggregate is used and the bitumen consists of cut-back asphalt or medium grade of tar. While many variations in the details of construction have occurred in the past, this work has now been more or less standardized in Indiana."

The process of applying the "retread top" was described by Mr. Hinkle thus:

"On top of the surface during any time from a week to six weeks after the oil is applied, crushed stone, slag, or gravel (preferably ranging in size from 1½ to 2½ in.) is spread on the surface from trucks to a depth of about 3½ in. This aggregate is leveled by the use of a heavy grader and the edges are trued up by hand. To this loose aggregate is applied about ½ gal. per sq. yd. of the medium tar or cut-back asphalt. The aggregate is then harrowed with a spike tooth harrow and struck off with a grader and rolled once immediately thereafter and as soon as the bituminous material hardens sufficiently so that the aggregate will compact and remain so, it is again rolled. A second application of bituminous material of about 0.4 gal. is applied and after curing, the surface is again rolled. Just preceding the third application of bituminous material, the voids in the coarse aggregate are filled with ¾-in. aggregate leaving a uniform and slightly excess layer of covering on the surface. After the third application of about 0.35 gal. of bituminous material the surface is planed or dragged and again thoroughly rolled. A fourth treatment of about 0.17 gal. per sq. yd. of the bituminous material covered by ½-in. aggregate may be used if necessary. This process resembles the mixing method in its first stages (except the size of aggregate) and its finishing operations are like the bituminous (penetration) macadam method. The smoothing and leveling are largely done mechanically and experience has shown that generally a smoother surface is produced and at a much less cost than an ordinary penetration macadam. The cost of this top course, 3 in. compacted, including the oiling of the old road for a 20-ft. width, in Indiana, has been from $3 800 to $6 000 per mile. Experience has shown that, where this top is built on a suitable base, it will carry any ordinary traffic that may be found on the average State highway. The surface is maintained by surface treatments as the ordinary bituminous macadam is maintained."

As an alternate to this method, said Mr. Hinkle,

"There is being advocated a plant mix method wherein aggregate about 2½ to 1½ in. in size, unheated, is coated with a medium grade of bituminous material before it is hauled to the road. This coated aggregate is spread upon the surface with the road grader and other machinery in much the same manner as the aggregate is spread in the road-mixing process. This coated aggregate is compacted by rolling after which a thin layer of ¾-in., coated aggregate is spread on the surface and the surface again rolled. There is now being experimented with, a system wherein this aggregate is coated by immersing it in the asphalt or tar as it is unloaded by conveyors from the car. These methods of making a plant mix of coarse aggregate bear about the same relation to the 'bituminous retread' previously described, as the plant oil-mixing method described by Mr. Frickstad bears to the road mix with the same grade of fine aggregate."

Commenting on types of equipment, Mr. Hinkle stated that in his opinion the long-base heavy drag or planer is the greatest development in the construction of the cheaper types of bituminous roads. With such equipment, he said,

"A surface can be made even smoother riding than by any hand methods yet devised. This smoothing applies especially to the oil-treated base for the 'retread top' where it is necessary to start producing a smooth surface if a smooth riding finished surface is obtained. Generally speaking, the equipment that is used in this type of work may be that used in maintaining an ordinary traffic-bound gravel or stone road. While special machinery may be provided, as Mr. Frickstad states they are doing in some places, fortunately this is not necessary for counties and other smaller governmental units which will have occasion to build short stretches of this type of surface."

Grading of Aggregate.—The method of applying the bituminous retread described by Mr. Hinkle, requires the use of a comparatively coarse aggregate as compared with the fine aggregate described in Mr. Frickstad's paper. In the opinion of Mr. Hinkle:

"If it is demonstrated that this fine aggregate will prove successful in those areas subject to deep frost action and heavy moisture, the fine aggregate method as described by Mr. Frickstad would prove of immense value in many areas where there is an abundance of local material of this character. The writer is in doubt, however, whether this fine aggregate can ever be made to support the loads in such areas that the coarser aggregate method is capable of carrying. In fact, with the coarser aggregate method the question seems to be largely one of sufficient base in order to carry the loads rather than a question of sufficient durability of the top. In some places this coarse aggregate method has been used on some of the heaviest traveled roads in Indiana and it can be said that, but for the necessity of treatments as a maintenance operation every year or so (which is some inconvenience to the traffic), the surface is holding satisfactorily."

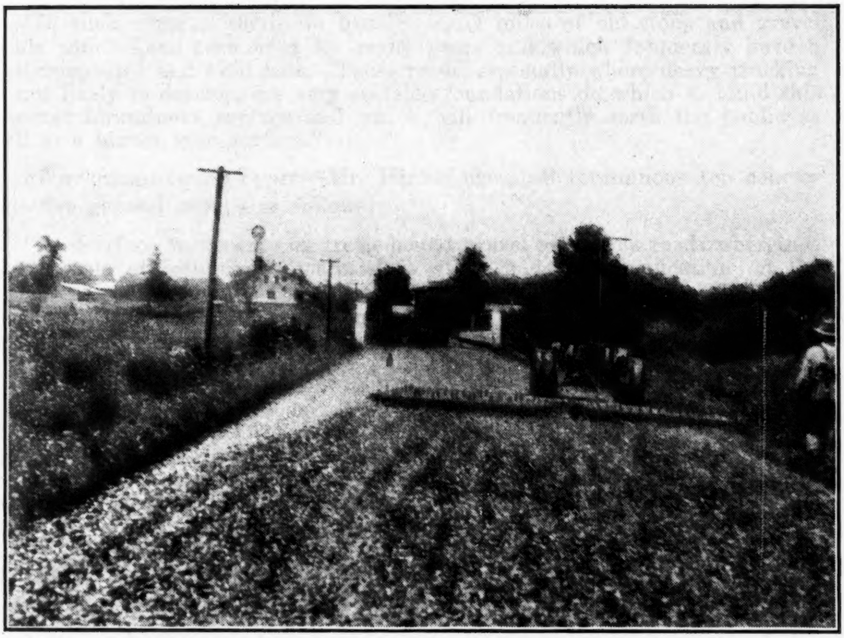
Bituminous Materials.—Defending his advocacy of cut-back asphalts and tars in this class of work, Mr. Hinkle explained further, that,

"Best results will be secured by using an asphalt having a penetration of 80 to 120 at 25° cent., and that it matters little whether this asphalt is put in a liquid form as a cut-back product or as a suitable emulsified product.

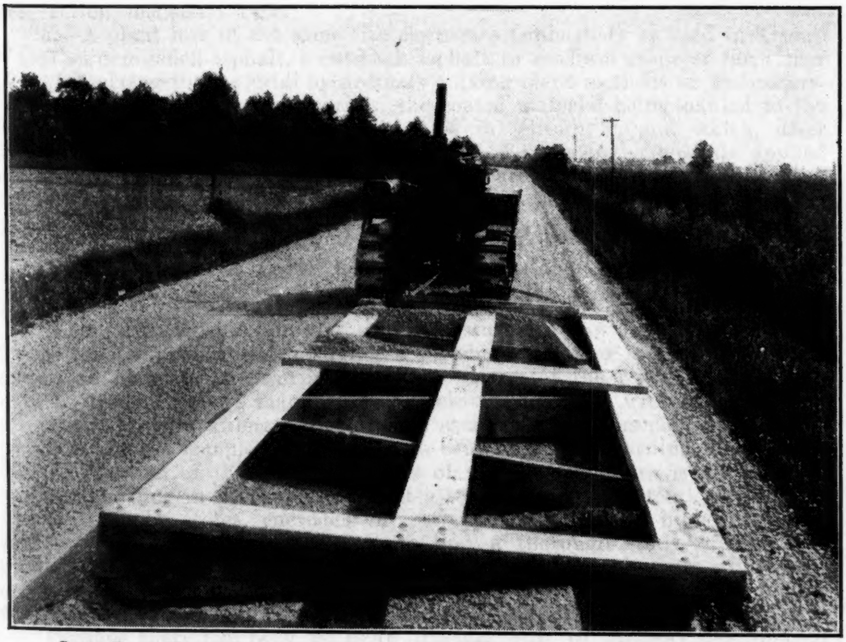
"While the fuel oil of California and other oils may not exactly comply with these requirements, they do not seriously depart therefrom. Also, the lower cost of the fuel and similar oils may more than justify their use even though they are not quite so satisfactory under certain conditions. With the use of this fine aggregate the first coat of bitumen at least must be a very light tar, fuel oil, or cut-back asphalt, with a certain kind of distillate, as other grades will not readily penetrate such aggregate.

"If further experiments with fuel oil or some suitable substitute mixed with aggregate containing an excess of fines proves successful in all climates, it will prove of great merit in those areas where much local fine aggregate is available, as frequently the local pits will supply this aggregate in about the proper proportions of fines that Mr. Frickstad describes as being used in California."

New Construction.—In a country subject to deep frost action and moisture, Mr. Hinkle expressed a doubt as to whether much new construction of this type would be done, because the cost of building an adequate base would be excessive. However, he said,



HARROWING ROAD SURFACE IN APPLYING THE "RETREAD TOP."



LONG-BASE HEAVY DRAG OR PLANER, USED IN CONSTRUCTING BITUMINOUS ROADS.

"In such climates there are usually many miles of old stone and gravel roads which have been used for many years and which frequently have a well-compacted and solid base. These roads, especially where heavy trucking is not likely to develop, are very suitable foundations on which to build this low-cost bituminous surface and which will frequently serve the public as well as a higher type surface."

Standardization of Types.—Mr. Hinkle classified bituminous top courses into five general groups as follows:

"1.—Surface treatments on traffic-bound gravel and stone roads wherein a light grade of bituminous material is used as a first application on the clean, compacted surface consisting largely of a fine aggregate, and then, by successive treatments with medium or heavy grades of bituminous materials and the addition of aggregate covering, a bituminous crust is built up sufficiently strong to carry the traffic.

"2.—The bituminous 'mulch' or 'oil mix' method wherein the light grade of bituminous material is mixed with fine aggregate on the surface to a depth of mixture of 1 in. to 4 in.

"3.—Coating the road with a plant mix of 1-in. to 3-in. depth, the mixture consisting of an aggregate containing a large percentage of fines, which mixture is spread on the road from a truck and leveled with a grader and other road machinery and maintained by blading and scraping until it hardens and is compacted by traffic (this mixture is virtually that described under 'Group 2,' but a plant mix instead of 'mixed in place.')

"4.—A bituminous 'retread top' made by mixing on the road an aggregate ranging in size from 2½ in. to 1 in. by successive applications of a cut-back asphalt, emulsified asphalt, or medium grade of tar, the first stages of this method resembling Group 2 method, and the final stages resembling the penetration macadam type.

"5.—A plant mix of the same size aggregate (unheated) as used in 'Group 4' and an emulsified asphalt, a cut-back asphalt, or medium grade of tar, either made by mixing the material by ordinary mixing-plant methods or by immersing it in such bituminous materials, the coated material being hauled to the road and spread in the same manner as in 'Group 3', and which, after thoroughly rolling and curing, is coated with ¾-in. coated aggregate applied and spread in the same manner, using a comparatively lean mix and sealing the surface with an ordinary seal or surface treatment coat if necessary."

The first three of these groups were described by Mr. Frickstad.

The Base Required Varies with Climatic and Soil Conditions.—In concluding his discussion, Mr. Hinkle explained that,

"Where deep frost action and excess moisture are prevalent, it is necessary to use greater precaution in having a suitable base than is necessary in the more arid districts and warmer climates. Many of the roads in California may be treated in the manner Mr. Frickstad describes without developing the base to greater strength, but if that same base were used in other areas, it would prove a complete failure. It is evident we are learning much about the construction of these cheaper types of surface and there is no doubt of their great value in developing at least a secondary road system. It is likely they will, as he states, encroach in many places upon the higher types of surface, yet it is extremely necessary that a thorough study of the local problems involved be made in order to avoid building such types where they are liable to prove failures and thus discredit them in their legitimate and economical use in other places.

"Perhaps there has been no more 'missing link' in highway development than the need of a low-cost secondary road which will carry a limited amount

of heavy traffic and an unlimited amount of light traffic, for it is evident that the bill for constructing the high-priced pavements on all highways will unnecessarily retard the proper development of any highway system."

GENERAL DISCUSSION

RIDING QUALITIES OF THE SUGGESTED SURFACES

By L. I. Hewes,* M. Am. Soc. C. E.

In stressing the importance of the subject presented for discussion, the Chairman, Mr. Hewes, emphasized the fact that little had been said of the riding qualities of the surfaces. For example,

"The travel in the West covers such long distances between cities that when one passes from the old dry gravel road on to the long stretch of the new type of work, and travels for scores of miles with scarcely any vibration of his machine, one senses a measure of the results that are being obtained."

Two factors were important, in the opinion of Mr. Hewes, namely, that the minimum width of roadway surfaces should be 20 ft. and that the bituminous mixtures must be placed dry. Furthermore, he questioned the fairness of imposing upon traffic the sole responsibility for rolling the unbound surface of a highway for 6 to 18 months. The rolling work should be part of the construction.

It seemed to Mr. Hewes that the application of the seal coat destroyed the smoothness of the surface, even when screenings were placed with the greatest care.

In closing the Morning Session, Chairman Hewes declared highway engineers have a remarkable state of affairs before them for study and analysis, namely,

"After fifty years of struggle with street asphalt mixtures we have passed into a new critical analysis of that type of surface which has revealed some remarkable new facts and some upsets. In general, however, it is agreed that the engineer must put in enough bitumen, or pure asphalt, not greatly in excess of the voids, and that he may increase the 200-mesh material. The result is that the optimum amount of asphalt is at least 8% or more."

The resulting stability then becomes a matter of concern, and to determine this stability, several types of standardized measuring machines are being developed.

Then, in 1927, a new mixture was introduced which, according to Mr. Hewes, required not much heat, nor much critical analysis of grading, and not much curbing of the material. A pavement top 3 in., or less, in depth is produced, which has remarkable stability.

Thus, on the one hand, said Mr. Hewes, there is the surface composed of 8% hard asphalt carefully heated and mixed with carefully graded material. On the other hand is the grading, roughly put together, with about 4.1% weak

* Deputy Chf. Engr., U. S. Bureau of Public Roads, San Francisco, Calif.

asphalt. The latter, according to Mr. Hewes, produces a road of remarkable stability, and yet it probably would be impossible to put a sample through the machine.

Afternoon Session—2:15 P. M. to 3:10 P. M.

WESTERN HIGHWAY PRACTICE

By C. S. Pope,* M. Am. Soc. C. E.

Writing under the caption, "Western Highway Practice with Special Reference to Some Construction Problems of the California Division of Highways," Mr. Pope devoted most of his paper to problems attending the protection of highways against destruction by storms.

The financing of highway work is now being planned on the assumption of an ever-changing, never completed program. Said Mr. Pope,

"Western highway engineers now realize that no highway system will ever be entirely completed, but that it will be added to, rebuilt and changed, and improved to the advancing needs of traffic farther into the future than any man can predict. Through the gas tax, a source of steady income, automatically adjusting itself to the finances required, has been set up, thus enabling construction programs projected from ten to fifty years into the future to be outlined with a moderate assurance that they will be carried forward. Whether a gas tax imposing a moderate tariff on motor vehicle owners will furnish sufficient funds to meet the demands of highway users, without the assistance of bond issues, is a question which time will answer."

However that may be, Mr. Pope declared that California is now allotting its definite road expenditures on the basis of the needs of the next ten years and is planning very much farther ahead than that in its progressive highway plans.

Clearing Before Construction

In many Western States, the clearing of ground to make way for highways is no small task. This may be judged from the costs presented by Mr. Pope. For example, he pointed out that to clear the right of way of moderate-sized trees and underbrush costs about $250 per acre. Trees in this class are most often uprooted by tractors. When the trees approach 18 ft. or more in diameter, however, they must be cut down by hand. The clearing cost may then become as high as $1 200 per acre.

On one project, said Mr. Pope,

"It was necessary to remove some 1 800 trees larger than 12 in. in diameter and ranging up to 18 ft. in diameter. The cost of clearing approximated $7.00 per thousand feet board measure of standing timber while stump removal averaged about $6.50 per stump."

* Chf. Constr. Engr., California Div. of Highways, Sacramento, Calif.

Then directing his attention at the subject of highway construction to furnish protection against the elements, Mr. Pope described in succession the structures used in protection against floods, cloudbursts, sea waves, moving sand dunes, and earthquakes.

Flood Protection

In arid regions where the most destructive floods occur considerable quantities of sand are scoured from beneath structures. Therefore, said Mr. Pope, solid structures are generally impracticable and the choice is usually for flexible types of construction or for those that can be easily replaced.

Brush and Wire Fencing.—Among these flexible types is the brush and wire fence, which, to quote Mr. Pope:

"Is constructed of two rows of galvanized-iron pipe driven deep into the sand parallel with the stream. Fencing of wire mesh is fixed to the pipe to retain the brush which is placed between the rows of fencing. The brush is sometimes weighted down with stones to insure its settling down and filling up any washouts which might occur under the fencing."

In this system, 2-in. pipes spaced 5 ft. apart are used. The two lines are 1.5 ft. to 5.0 ft. apart and the space between is filled with alternate layers of willow brush and stone. Mr. Pope declared that a 2½-ft. layer of brush would be compacted to 1 ft. under these conditions. In a variation of this method, sausage-like containers of wire mesh are filled with stone and placed in piles between the two rows of pipe.

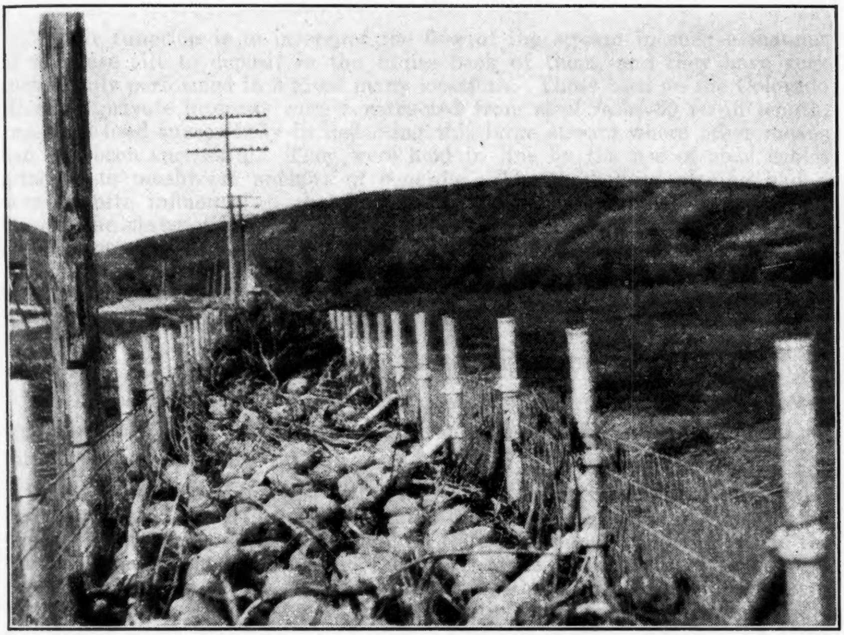
Another form of flood protection described by Mr. Pope was the slope mat of wire mesh and cobbles which to a certain extent will,

"Follow any undulations which may occur due to washouts. It has been extensively used in storm protection work in Los Angeles County, but not to any great extent by the Highway Organization. The cost is stated as about 25 cents per sq. ft."

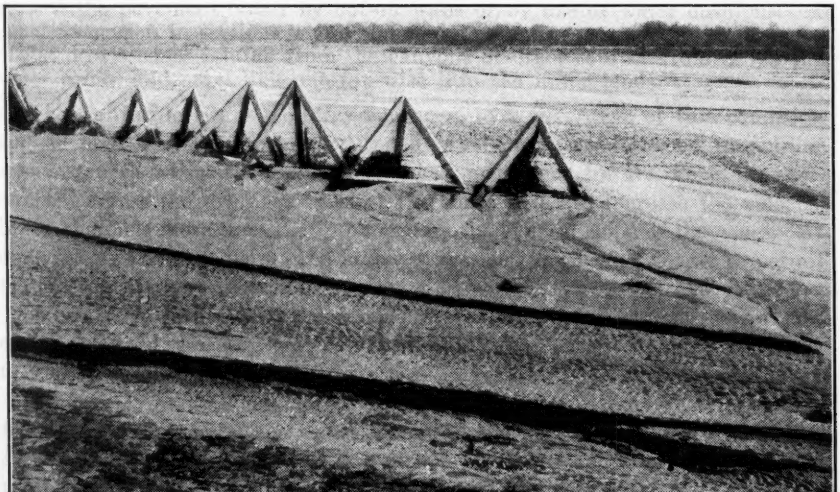
Reinforced Concrete Slope Paving.—Concrete paving is used extensively for bank protection in California. Describing the elements of the process, Mr. Pope explained that,

"It is usual to place a suitable toe wall of sufficient depth to guarantee the paving against scour. The considerations governing the thickness and reinforcing details of the slope paving are the slope height and the character of the detritus carried by the stream. In some localities it has been found desirable to make the thickness of slope walls more than 6 in. to resist the action of large boulders brought down by floods, but in most cases a thickness of 6 in. is sufficient. Banks of medium height are protected by a 4-in. slab, or by the use of 2 in. of gunite. The cost of reinforced concrete slope paving varies with the local conditions."

Jetties.—In the northern part of California, where the stream beds are generally more stable, deflecting jetties are sometimes constructed of piles with wire mesh or barbed wire nailed to them. In places where the detritus carried by the stream is rather light skeleton tetrahedrons of steel or concrete have been found very effective for stream deflection and bank protection, according to Mr. Pope. Describing these tetrahedrons, Mr. Pope said that:



Dike of Wire Fencing and Brush Used as Stream Deflector, Southern California.



Reinforced Concrete Tetrahedrons Used as Stream Deflectors and to Build up Shore Line in Stream.

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"Their function is to interrupt the flow of the stream in such a manner as to cause silt to deposit in the eddies back of them, and they have very successfully performed in a great many locations. Those used on the Colorado River by private interests were constructed from steel rails, 30 ft. in length, and were used successfully in deflecting this large stream where other means had not been successful. They were held in line by the use of steel cables attached to mushroom anchors of concrete. The method of placing had a very definite influence on their successful use. This method consisted of placing the obstructions one after another down stream, which caused silting of the protective bank to begin at once and prevented further scour.

"On highway work a row of reinforced concrete tetrahedrons which had been placed to deflect the stream on the Ventura River was the means of saving a considerable section of highway during the St. Francis Dam disaster."

Protection Against Cloudbursts

Mr. Pope asserted that engineers generally feel that cloudbursts recur approximately within the same areas from time to time. His comment on this topic may be quoted in full:

"The water often appears without previous intimation that a cloudburst has occurred. The first knowledge which the observer has of the approaching danger is the appearance of a wall of water and mud sweeping down the canyon. The method pursued in the past had been to construct paved dips across all locations where the profile indicated that storm or cloudburst run-offs were usual. This, however, proved unsatisfactory in many cases because of extreme scour which occurred at the overflow aprons, and also the large expense of cleaning débris from the dips and roadway after each storm. It seemed practically impossible to check the velocity of the water after it had crossed the pavement either by cut-off walls or by use of water cushions. A usual formation in sections where cloudbursts are frequent shows the presence of flat débris cones issuing from the canyons or other sources from which the water comes and spreading out fan-wise into the lower lands."

The California State Highway engineers took advantage of this condition and adopted a system formerly used by the Santa Fé Railroad in similar circumstances. Pick-up channels or dikes, mentioned by Mr. Pope, are constructed across the drainage lines of the cones so as to increase the velocity in the channels more than that produced by the general slope of the cones. At some selected control points the highway is constructed on a trestle to permit the passing of the stream.

When wide open valleys are encountered in desert regions, storm flow occurs as broad sheets of water. Then, said Mr. Pope, it becomes necessary to construct extensive dike systems.

The costs for this class of protection work were given by Mr. Pope, as follows: Ditching on débris cones, about \$1 500 per mile; wider ditches and dikes for wide desert valleys, about \$2 500 per mile; and timber bridges and trestles with creosoted piles, 20 ft. to 24 ft. long, and a 24-ft. roadway superstructure of redwood, from \$100 to \$125 per ft. of length. In general, highway protection of this type will cost from \$3 000 to \$3 800 per mile.

Sea Protection

In a number of places in California, highways have been located along the sea and, consequently, according to Mr. Pope, it has been necessary to provide

structures for their protection. Booms chained to piling are a temporary expedient, he said, and this method is not now used in highway work.

Protection Work at Santa Monica.—A very interesting description of special problems encountered in this class of work was presented by Mr. Pope. Quoting:

"Random rip-rap with the majority of stones weighing not less than 5 tons has been used to some extent on highway work where the action of the waves and tides is normal to the coast line; but the type of equipment available on highway work makes the handling of heavy stones extremely difficult. This situation arose in connection with certain work in Southern California along the rocky coast north of Santa Monica where protection of slopes against sea action became necessary. It was decided that since large rock suitable for both rip-rap and blanketing the slopes could not be handled at the site of the work that a different plan of protection should be used. Accordingly, a design was worked out under which concrete cells should be constructed which could be lowered into place and afterward filled with concrete to increase their weight.

"The reinforced concrete cells were 10 ft. long, 5 ft. wide, and 3 ft. high, and their estimated weight was about 3½ tons before filling, or 12 tons in place.

"Their cost in place was about $95 each, or approximately $8 per ton.

"After the cells had been set in place and before they were filled with concrete, it was necessary to grout the boulders on which they were set with quick-setting concrete in which cement and hard wall plaster were combined in proportions of 1 to 1 in order to insure sealing of the concrete against wash from below. When the cells had been filled completely, it was again necessary to protect the fresh concrete against wave wash by boarding the top.

"The shore line at this location is entirely exposed to the full sweep of the Pacific Ocean and consisted of rocky bluffs with a steep slope into deep water. The design in this case called for the construction of a toe wall below high tide, the paving of the slope in front of the toe wall to below low water, and the construction of reinforced concrete slope paving 9 in. thick based on the toe wall and ending in a parapet designed to throw back running waves. It was possible to place the concrete cells along the tide line, but owing to the heavy run of the sea, it would have been impossible to have built the toe wall or the slope walls without some protection. Accordingly, such leveling and removal of boulders as was possible was done at low tide and the concrete cells were then sunk into line on the roughly prepared foundation and filled with concrete and heavy stone between tides.

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"The cells being poured and the beach boulders grouted, enough protection was afforded so that sufficient time was allowed in spite of the rising tide, to pour the base of the toe wall to a height of 1½ to 2 ft., leaving from 1½ to 2½ ft. to form on top of the base, the boxes acting as a form on the outside. The section of base being poured, the top section was poured into the forms built on the base poured previously. These forms were built while the other work was proceeding. The toe wall was poured with Class "C" concrete, 4.2 sacks per cu. yd. Steel bars, ½ in. by 4 ft., were used in the toe wall. These bars were embedded in the base at 12 in. centers, extending up through and out the top of the wall to tie the toe wall to the slope paving.

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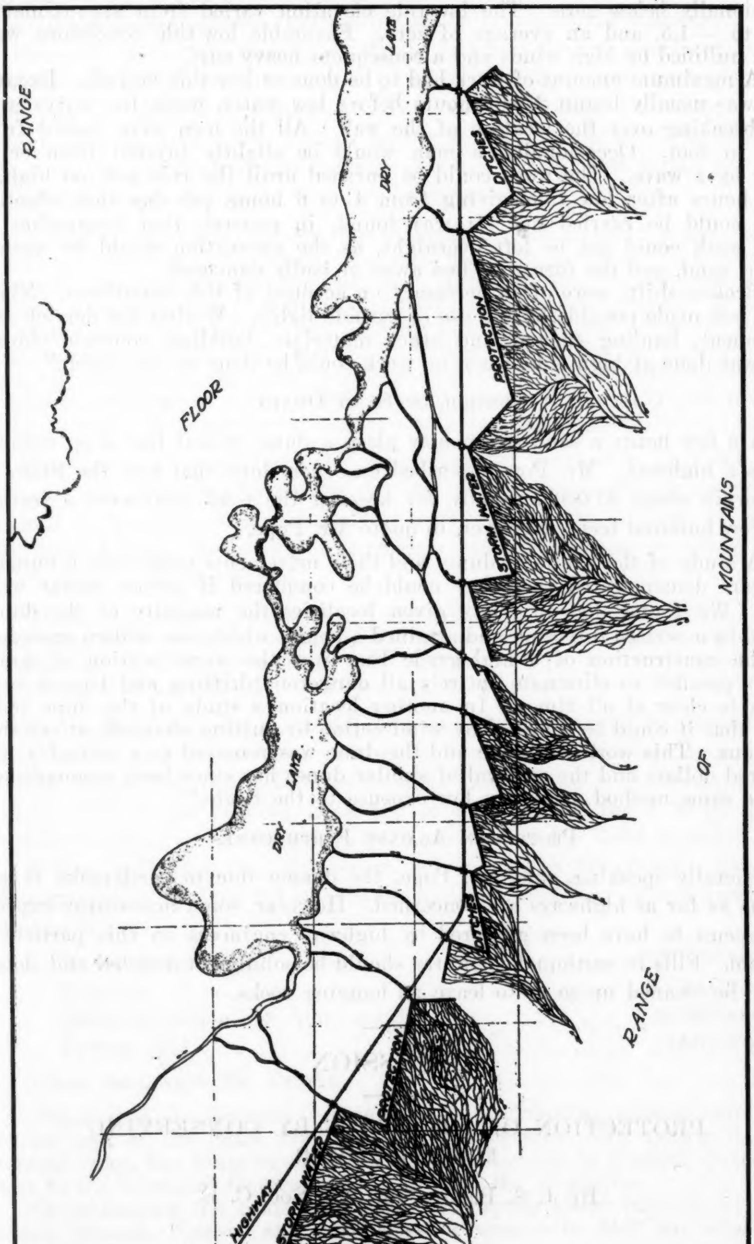
"This work had to be carried on during the winter months, the most unfavorable time of the year, in order not to delay the slope paving. The work was all at a low elevation on the beach, varying from plus 3 to 0, and



TYPICAL HIGHWAY LOCATION, PARALLEL TO A DESERT VALLEY, WITH STORM PROTECTION DITCHES.

occasionally below zero. The low-tide elevation varied from approximately +2 to —1.5, and an average of zero. Favorable low-tide conditions were often nullified by high winds and a consequent heavy surf.

"A maximum amount of work had to be done at low tide periods. Excavation was usually begun 2 to 3 hours before low water, while the waves were still breaking over the location of the wall. All the men were soaked from head to foot. Occasionally, a man would be slightly injured from being upset by a wave. The work could be pursued until the tide got too high, 2 to 3 hours after low tide, giving from 4 to 6 hours per day that effective work could be carried on. It was found, in general, that excavation or form work could not be left overnight, as the excavation would be washed full of sand, and the forms washed away or badly damaged.

"Broken shifts were often necessary on account of tide conditions. Night work was made possible by the use of carbide lights. Moving the derrick and equipment, hauling cement and other materials, building concrete chutes, etc., was done at high tide, when no work could be done on the beach."

Control of Sand Dunes

In a few hours a sand storm may place a dune several feet deep directly across a highway. Mr. Pope described one such dune that cost the State of California about $7 000 annually for keeping the road clear over a section of a few hundred feet. However, to quote Mr. Pope,

"A study of the habits of dunes and their movements made over a number of years demonstrated that they could be conquered if proper means were used. We found that in many given localities the majority of the dunes moved in a certain direction and attained a height which was seldom exceeded. By the construction of a high-grade line over the worst section of dunes it was possible to eliminate entirely all dangerous drifting and have a road which is clear at all times. In another location a study of the dune indicated that it could be removed by wind action by cutting channels at suitable locations. This work was done and the dune was removed at a cost of a few hundred dollars and the removal of similar dunes has since been accomplished by the same method at a very low expense to the State."

Protection Against Earthquakes

Generally speaking, said Mr. Pope, the danger due to earthquake is not serious as far as highways are concerned. However, some noteworthy experience seems to have been gathered by highway engineers on this particular problem. Fills in earthquake country should be solidly constructed and slopes should be cleaned up so as to leave no hanging rocks.

DISCUSSION

PROTECTION OF HIGHWAYS BY CONSERVING FOREST COVER

By J. S. Bright,* M. Am. Soc. C. E.

Two factors of highway construction which contribute to erosion at some later date, in the opinion of Mr. Bright, are the destruction of the forest cover

* Constr. Engr., U. S. Bureau of Public Roads, San Francisco, Calif.

and the filling of watercourses with large quantities of construction débris. In addition, these tendencies destroy the scenic beauty of the highway.

Recently, the U. S. Bureau of Public Roads, with the assistance of the National Park Service, has taken steps to protect the forest cover along the right of way during construction. The best protection, according to Mr. Bright, has resulted from designs involving full bench sections, balanced fills, tunnels, hand-laid embankments, concrete cribs, and retaining walls.

Specifications.—To quote Mr. Bright directly:

"Specifications have provided that clearing shall be done in stages, leaving a screen of timber on the lower side until after the heavy shooting has been finished. The blasting requirements have stipulated that the contractor shall furnish a definite plan of drilling and loading, for prior approval. It is contemplated that the engineer will approve this plan on its successful demonstration over a small area. The 'teeth' in the specification give the engineer authority to require the spacing of holes at not to exceed 2-ft. centers, with a minimum charge of ½ lb. of 40% strength dynamite per cu. yd. of material to be moved. Mats are to be required as a last resort."

Penalties are imposed on the contractor for infraction of these rules. For example, according to Mr. Bright, the contractor's work may be stopped; he may be required to gather up the scattered material or trim or remove any injured trees. As a result, contractors have devised the necessary means of protection. Among these,

"Hand-laid embankments have proved satisfactory and have involved a cost from $1.40 to $2.00 per cu. yd. over and above the usual cost of rock excavation. Reinforced concrete cribs are adapted to the same purpose, and have been costing from $2 to $6 per sq. ft. of exposed face."

In Mr. Bright's opinion, tunnels are ideal in preventing the detraction of forest cover, and many short ones have been built. The tendency is now toward greater lengths. Highway tunnels have proved economical, said Mr. Bright, "the prices varying from $55 to $100 per ft. for a 23-ft. unlined roadway section, and from $80 to $130 per lin. ft. for lined tunnels."

Finance.—During the 1928 season, as reported by Mr. Bright, the eleven Western States received $91 500 000 for highway purposes. The sources given were as follows:

Motor fuel tax	$40 000 000
Licenses	18 000 000
Other miscellaneous State sources	18 500 000
Federal Aid	15 000 000

These data, said Mr. Bright,

"Show that the Federal Government is contributing approximately 16% of the cost of the State road systems in this area. Or, taking it in the reverse order, the State systems are being united into a Federal system at a cost to the National Government of 16% of the programs.

"Furthermore, the State road users are paying about 76% of the State's share, through licenses and fuel taxes, leaving only 24% for other State interests to contribute.

"With such a large contribution by the road users, it is apparent that road finance budgeting should give first consideration to the serving of

traffic. There is nothing fiscally unsound in stage construction if practiced with the major purpose of serving traffic.

"This stage construction may involve narrow surfaces, pioneer roads with short stretches of steep grades, concrete fords in lieu of expensive structures, and temporary oil process surfaces on old county roads. This oil-processed material can be readily salvaged and transported to the final location. The oil processing of base courses for serving traffic is also an economical expenditure where mileage and short seasons are governing factors.

"A well-planned budget is paramount for good administration. It should cover a series of years, letting the improvements advance in an orderly manner. Service to the public should demand that budgeting prevent dead-end projects, or large sums being tied up in frozen surveys."

Mr. Bright expressed approval of Mr. Pope's suggestions for clearing right of way well in advance of construction. In his opinion it was possible by this means to do any burning necessary when fire hazards are least. Furthermore, slope stakes can thus be placed far enough in advance of construction and the cleared right of way permits the sun to melt the snows, thus lengthening the construction season.

DISADVANTAGES OF LONG-TERM FINANCING

By J. M. Howe,* M. Am. Soc. C. E.

In discussing the financing phase of Mr. Pope's paper, Mr. Howe called attention to a few arguments against the suggestions therein. At present, he said, there is a proposal being discussed in Texas to raise $300 000 000 by bond issue, with the idea of spending it on State highways during the next ten years. Opponents of the bond issue are arguing that road types are changing so rapidly that much work built at once would become obsolete before the 10-year program is completed.

Mr. Howe was inclined to question the policy of raising a very large bond issue for long-term financing, on the ground that one poor State administration could dissipate the funds.

Influence of Soil Classification.—Another factor that might contribute toward obsolescence in highway construction of the future was research in the field of soil analysis. Citing the pioneer work of Charles Terzaghi, M. Am. Soc. C. E., in this field, Mr. Howe called attention to the possibilities of the future as applied to highways, especially road foundations.

For example, he cited the expressed opinion of an eminent engineer who stated that the investigation of colloids and the work of physicists would probably develop a method of hardening or toughening various soils. In summing up these thoughts on factors that might produce early obsolescence, Mr. Howe stated that such developments would cause a complete change of front in highway construction methods. The work of bank protection, he said, might be thus made almost unnecessary, and wearing surfaces would probably become thinner.

* Cons. Engr. (Howe & Wise), Houston, Tex.

IRRIGATION DIVISION

April 24, 1930—8:40 A. M. to 12:00 M.

THE PROPOSED COLORADO RIVER AQUEDUCT AND METROPOLITAN WATER DISTRICT

By Frank E. Weymouth,* M. Am. Soc. C. E.

After a preliminary business session, the paper by Mr. Weymouth was read by Julian Hinds, M. Am. Soc. C. E. It began with an outline of the specific problems encountered in satisfying the increasing demands for water in Southern California and particularly in the general vicinity of Los Angeles. For many years wells were found to be adequate; but as the water needs increased withdrawals exceeded replenishments to such an extent that artesian wells ceased to flow and ground-water levels receded.

Twenty years ago, according to Mr. Weymouth, the City of Los Angeles began to feel the inadequacy of its local supplies, with the result that the aqueduct tapping the Owens River, 250 miles north of the city, was constructed. It was designed to deliver about 400 sec-ft. This supply has now been almost entirely absorbed and a new supply must be sought.

That this situation is general was emphasized by Mr. Weymouth, as follows:

"Practically all the other cities in the district have had similar experiences. Supplies once thought adequate have been exhausted, or have dwindled, due to the over-pumping of underground basins. New supplies brought in are soon absorbed by the rapidly growing communities. A point has been reached where stagnation can be averted only by promptly bringing in a large new supply of water. The only practicable and adequate source of such a supply is the Colorado River."

At its nearest point, the Colorado River is about 210 miles from Los Angeles. The problems attendant upon building the proposed aqueduct are such as to require the co-operation of a number of cities before the main supply line can be built. To this end, the Metropolitan Water District of Southern California was formed in 1928 and eleven cities have entered the District to date.

All plans for this aqueduct are based on a maximum capacity of 1 500 sec-ft., continuous flow which, according to Mr. Weymouth, would still be insufficient for the ultimate expected demands, but it is the maximum available water.

Mr. Weymouth discussed in general terms the formidable problem of selecting the best route.

* Chf. Engr., Metropolitan Water Dist., Los Angeles, Calif.

"The diversion," he said, "may be made almost anywhere from Bridge Canyon to Yuma and from the several most promising intake points there are numerous alternative routes. Choice must be made between comparatively straight and short tunnel lines and longer surface conduits, and between pumping lines and gravity lines."

The basic problem resolves itself into finding:

"The line along which an aqueduct may be constructed which will deliver water to the various cities of the District at a minimum cost per acre-foot, keeping in mind that the aqueduct must be physically and economically feasible. In the solution of this problem, more than sixty-five separate preliminary aqueduct lines have been projected with various diversion points from Bridge Canyon to Yuma."

A Lower River Line

The shortest and most direct route was stated by Mr. Weymouth to be obtained by constructing an aqueduct diverting at a point near Yuma, Ariz. Describing the advantages of this route, Mr. Weymouth wrote:

"This line contemplates pumping, either directly from the river, or from a low diversion dam at Picacho, a point about 25 miles north of Yuma. Delivery is to be made at Elevation 1 000 into a proposed enlargement of the existing County Flood Control Reservoir at Puddingstone, about 30 miles east of Los Angeles.

* * * * * * * * *

"At the river the water is elevated only sufficiently to bring it out on to the floor of the Imperial Valley. The principal lift is made at the foot of the mountains near Indio, to gain the elevation needed through the San Gorgonio Pass. Power for pumping is to be transmitted from Boulder Canyon. Because of the low elevation of the water in the river the net pumping, that is, the total lift less return power, will be greater for this line than for lines diverting at higher points. Also, because of the distance of the point of diversion below Boulder Canyon, it is to be expected that present silt conditions will continue for some years, making it necessary to build and temporarily operate a desilting plant.

"These disadvantages are compensated for by a substantial reduction in the construction cost."

Pumping Line from Boulder Canyon

Another route is based on a proposal to divert directly from the Boulder Canyon Reservoir. This route, said Mr. Weymouth,

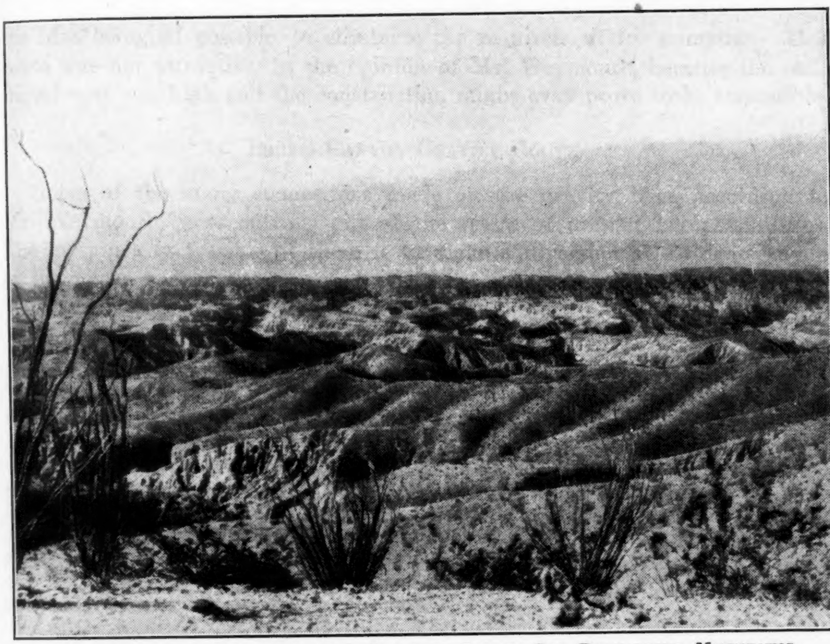
"Profits by the increased elevation of the water surface, and by the complete elimination of any trouble from silt in the water. The necessity for long transmission lines is also eliminated. The principal disadvantages of such a line are increased length and the ruggedness of the area over which it must be constructed."

Furthermore, in considering this route,

"The selection of the proper height of pump lift is also of great importance. A balance must be secured between the cost of power, carrying charges, and the ability of the community to meet investment costs. This line is longer than any other of the lines shown, but it is not necessarily the least desirable."

Gravity Route from Boulder Canyon

One of the proposals described by Mr. Weymouth was to construct a single straight-line tunnel from Boulder Canyon to a point near Monrovia, Calif.,



"BAD LANDS" ALONG SOUTHWEST SLOPE OF LITTLE SAN BERNARDINO MOUNTAINS, SALTON SEA IN UPPER LEFT.

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the idea being, if possible, to eliminate the requirement for pumping. This route was not attractive, in the opinion of Mr. Weymouth, because the estimated cost was high and the construction might even prove to be impossible.

Bridge Canyon Gravity Route

Some of the many suggestions made on the gravity idea, according to Mr. Weymouth, were entirely out of the realm of present-day possibilities. The most nearly feasible of these is to make a diversion at Bridge Canyon at the upper end of the proposed Boulder Canyon Reservoir. In part, Mr. Weymouth's comments on this route are as follows:

"Its total length as projected is 284 miles, being made up of 231 miles of grade tunnel, 46 miles of surface conduit, and 7 miles of pressure tunnel. The longest tunnel on the line is 104 miles. Many deep shafts are required for its construction. This line has the advantage of delivering water by gravity at a usable elevation, and it will yield an appreciable block of power at Bridge Canyon. However, the Bridge Canyon Reservoir capacity is too small to control the river effectively, and power output would not be as great as the height of dam might indicate. The deep tunnels of this route cannot entirely avoid passing underneath the débris-filled, water-logged basins of extinct inland seas, and serious water troubles are to be anticipated in its construction. It is doubtful if a location entirely in rock could be secured with any reasonable tunnel alignment."

Routes Now Under Study

There are many possible variations of the routes described. For example, quoting from the paper,

"The gravity routes may be varied as to location, height of diversion, size and shape of conduits, gradients, points of delivery, etc. In the pumping routes the height of lift is an additional variable. The lower river diversions also involve the possibility of power development at the point of diversion."

Conduit and Tunnel Sections

The preliminary studies contemplate the use of concrete-lined tunnels. Describing these details further, Mr. Weymouth stated that,

"Generally, the tunnels are planned to run free, following the hydraulic grade. Pressure tunnels are proposed only for possible deep river crossings, and as an alternative in the case of an all-tunnel gravity route from Boulder Canyon. All surface conduits are to be of the cut-and-cover type. The section at present contemplated is of modified horse-shoe shape, and is to be entirely buried. Reinforcing is contemplated only at points requiring special treatment. For crossing depressions, concrete and steel pipes, as well as pressure tunnels, are being considered."

Quoting further,

"In some places a reduction in cost might be effected by the use of an open canal in place of the proposed covered conduit. However, an open ditch for the conveyance of domestic water is objectionable, especially through cloudburst and sand dune areas where breaks are likely to be frequent. The open ditch exposes the water to pollution, encourages aquatic growths, and involves added losses through seepage and evaporation."

Quality of Water

Mr. Weymouth pointed out that various possibilities had been widely discussed as to the effect of silt, rock salt, etc., on the water to be used for domestic and industrial purposes. To these questions Mr. Weymouth stated that,

"All reports from reliable sources show the danger of salt contamination from deposits in the proposed reservoir area to be negligible. * * * Water from the river is now in use by the City of Yuma, Ariz., various Imperial Valley municipalities, and a number of irrigation projects. The water is believed to be entirely suitable for the uses of the Metropolitan Water District, and better than the supplies of many cities."

Accomplishments to Date

Mr. Weymouth outlined the present status of the project as follows:

"Topography has been taken over approximately 30 000 sq. miles of desert and mountain area; various dam sites on the Colorado River have been surveyed and tested; terminal reservoir possibilities have been investigated; geological conditions over a wide area have been studied in detail; projections and estimates for more than sixty-five separate preliminary routes have been made; many data have been gathered on the quality of the water and methods of clarification; preliminary plans have been made of many important features; and financial problems have been closely studied.

"Preliminary arrangements for a power contract have been made, assuring to the District all the power needed for pumping, at a price sufficiently low to make the project feasible.

"In December, 1929, the available data were laid before a Board of Consulting Engineers, and after due consideration of the then known facts, that Board made definite recommendations as to the additional data required before a final selection of route can be made. The additional tests, surveys, and studies requested are being made, and will soon be completed, when the facts will again be submitted. It is hoped a final selection of a route can be made and arrangement for a bond election completed before the end of 1930.

"In conclusion, it may be stated that every possible effort is being made to view the problem in its broadest and most fundamental aspects. It would be very desirable, if practicable, to eliminate all pumping and otherwise to reduce the annual expenditures required after the bonded indebtedness is retired, and to give to the future a practically free water supply. However, it would be foolish to bankrupt the present community in a futile attempt to meet this ideal. From a purely mathematical point of view, the best solution is that which gives the lowest present value of all combined present and future expenditures, but the pure economics are upset by the legal requirements of repayment, and by the prospective increase in the financial ability of the associated cities of the District as time goes forward. It is this latter factor that lends favor to the pumping projects, as against the more expensive gravity routes. To attempt too great a present burden might create such an unfavorable condition as to cause the growth of population to fall short of the present optimistic estimates, thus delaying indefinitely the development needed to use and pay for the aqueduct flow. The aqueduct is to be built not for the present, but for the future, and future generations must be willing to bear whatever part of the burden cannot be borne now. The proposed aqueduct is not of a transitory nature, like many other public improvements, but may be expected to endure indefinitely. Nevertheless, it must be financed under the same schedule of bond repayment as other comparatively temporary

improvements. No part of the cost of construction can be legally deferred for repayment after the full need for the water has developed. Therefore, if the present cost of the physically most desirable project is prohibitive, some compromise plan that can be safely financed must be adopted. If such a plan requires pumping, then the payment for power will be posterity's contribution to the project. It must be recognized, of course, that the future will bring with it new problems, and we have no right to bequeath to coming generations any unnecessary or impossible burdens. A solution must be sought which will be fair and equitable, and practicable in all its phases."

DISCUSSION

REVIEW OF THE PROBLEMS INVOLVED

By Louis C. Hill,* M. Am. Soc. C. E.

A general summing up of the salient points in Mr. Weymouth's paper was the purpose of the discussion by Mr. Hill. In this vein he stated,

"After Mono Basin water has been added to Los Angeles' present supply there will probably be ample water for that city for the next ten to fifteen years. The other cities included in the District are, however, not so well supplied, so that even if Boulder Dam be not built, it will be necessary for the future growth of Southern California to bring water from the Colorado within a few years. No other adequate source exists within a distance economically feasible.

"The Colorado River Board has very recently recommended raising Boulder Dam about 25 ft., or to a roadway elevation of 1 232 ft. above sea level. When completed the dam will be about 715 ft. high and the resulting reservoir will have a capacity of about 30 000 000 acre-ft., or more than ten times that of the largest artificial reservoir in the United States. This reservoir will reduce the floods in the Colorado from more than 25 000 cu. ft. per sec., or from the flow at Niagara, to about 60 000 cu. ft. per sec., and they would reach this amount only about once in 500 years.

"According to the Swing-Johnson Bill, Congress will make available the money for the construction of Boulder Dam only after contracts for the sale of power to be generated at the dam shall be signed, which contracts will in the opinion of the Secretary of the Interior repay the United States for its expenditures, together with interest thereon at 4%, in 50 years.

"About 1 000 000 h. p. will be installed capable of delivering to the high-tension lines about 4 000 000 000 kw. of firm energy and nearly half that amount of secondary energy each year."

Power Required to Pump Into the Aqueduct.—If the route beginning at the aqueduct is adopted the water must be lifted about 1 700 ft., according to Mr. Hill. This, he said, will require about 375 000 h. p. in motors to drive the pumps. To lift the 1 100 000 acre-ft. over the summit of the aqueduct would require about 2 500 000 000 kw-hr. Mr. Hill gave the total length of this route roughly as 310 miles, including from 80 to 100 miles of tunnels.

An alternate line (referred to by Mr. Weymouth as Route No. 1) is 215 miles long, including 70 miles of tunnel. Of this proposal, Mr. Hill stated,

* Quinton, Code & Hill—Leeds & Barnard), Los Angeles, Calif.

"The lift is about 1 340 ft., or if Picacho Dam is built about 1 300 ft. The horse-power and energy required would be about 80% of that required if Route No. 2 be adopted. The transmission line would be more than 100 miles longer. However, there are many things besides power consumption and length of line which will finally enter the equation before final selection is made.

"The lower down the river the diversion, the more power can be developed by the water ultimately to be diverted. If diversion takes place at Boulder Dam about 80 000 h. p. will be lost, or enough power to lift the aqueduct water nearly 500 ft.

"Water diverted at Boulder Dam, or from Bulls Head Reservoir, will be completely desilted. If diverted below it must be desilted by some means preferably by storage in a re-regulating reservoir. Temporary methods should be adopted at least for Picacho Reservoir until the river has had time to clear itself after which the by-pass may be closed. It would probably require from 10 to 15 years after Boulder Dam is finished, but the water pumped into the aqueduct during this period would be but a small portion of its capacity.

"Power could be developed at any of the dams below Boulder Dam. After the latter is completed earth-fill dams could safely be built at any of the suggested sites where by-passes and spillway can be located in rock.

"*Faults.*—All routes across the great San Andreas fault, and most of the routes cross several other faults.

"If possible all active faults at least should be crossed at surface level. To provide for possible movements along these faults the profile of the aqueduct should show quite large drops at each fault—the magnitude of the probable movement at the fault determining the amount of the drop. Plenty should be provided. If then movement does occur the result after repairs will usually be to either increase or decrease the drop.

"All lines apparently go through either Cajon Pass or San Gorgonia; the latter pass, if the line crosses the valley, is the better geologically.

"As the author has well stated, the problem is not alone one of picking the route which will ultimately deliver water at the least cost per thousand gallons. The ability of the District to finance the enterprise enters largely into the equation.

"A study is possibly being made to determine the cost of first building the aqueduct to part capacity and later, when more water is needed, enlarging it to full capacity or of building another aqueduct.

"The greater cost of such a course would possibly be offset by the saving in interest and depreciation, augmented by the better financial set-up especially as posterity would then have to bear its share of the burden.

"*Storage.*—Again, since the Los Angeles Aqueduct and all proposed lines from the Colorado River cross the great San Andreas fault as well as several lesser ones and since a major movement along any one of these faults might put the aqueduct out of commission for at least three months, there should therefore be sufficient storage provided (in addition to such seasonal storage for operating as is necessary) within or contiguous to the Metropolitan District to supply the District for at least that three months. The total reserve storage held for emergencies should therefore equal about 25% of all foreign water, or nearly 300 000 acre-ft."

GENERAL DISCUSSION

That the problem of lifting 1 500 sec-ft. of water 1 600 ft. is a real undertaking was emphasized by Richard R. Lyman, M. Am. Soc. C. E. He

expressed doubts as to whether the Engineering Profession has given consideration to a more difficult problem, and took occasion to compliment those who are connected with it.

Several significant aspects of the Colorado Aqueduct enterprise were emphasized by Franklin Thomas, M. Am. Soc. C. E. One was the fact that the project is bringing together a number of communities for the solution of a common problem. The other is that, although the City of Los Angeles could have built the aqueduct for its own consumption, it was willing to share the benefits with the satellite communities of that area.

Of the need for the improvement, Professor Thomas said:

"Some of the smaller cities and communities within the area of the Metropolitan Water District are faced with a continually receding water-basin level. It is estimated by the Division of Water Rights of the State of California that this total prospective metropolitan water area now is overdrawing the available supplies of water to the extent of 260 cu. ft. per sec., continually. This overdraft is anticipated to reach 600 sec-ft. in about ten years, and, by 1956, would aggregate, according to present trends, an overdraft equal to the amount of imported water."

At present, the Metropolitan Water District includes eleven communities. Professor Thomas expressed the hope that ultimately the entire area and all the communities within the region south of the San Gabriel Mountains and west of the San Gorgonio Divide would share in the costs and benefits.

Commenting further on the seriousness of the situation, Professor Thomas remarked,

"In a number of these communities the ground-water levels are receding at about the rate of 1 ft. per month. This, of course, is really a critical condition. It is not known just where the limit may be reached, and with reference to the communities nearer the shore line, it is a question as to when there will be a reversal of flow, or a landward movement of sea water. It is hoped that when that newly imported supply is brought in, meteorological conditions will improve by a series of full years of precipitation so that the worst catastrophe may be averted."

FOUNDATION TREATMENT OF THE RODRIGUEZ DAM ON THE TIJUANA RIVER, MEXICO

By Charles P. Williams,* M. Am. Soc. C. E.

This paper contains an account of the design and construction of the foundation structure for the Rodriguez Dam which is at present being built on the Tijuana River at Baja California, Mexico. As described by Mr. Williams, the purpose of this dam,

"Is the storage and diversion of water for the irrigation of about 5 000 acres in the Tijuana Valley in Mexico, and for domestic supply for the municipality of Tijuana now having a population of 10 000.

"The drainage basin of the Tijuana River, which lies partly in Baja California and partly in San Diego County, California, has an area of 1 670 sq.

* Cons. Engr., Los Angeles, Calif.

miles, which is greater than that of any other stream south of Los Angeles and west of the Colorado River; it nearly equals the total area of the drainage basins of all other streams in San Diego County. The stream has two principal branches, upon the larger and more southerly of which the Rodriguez Dam is being constructed. The area of the drainage basin above the Rodriguez site is 938 sq. miles."

In ordinary years the flow of the Tijuana River is very small, and the supply is necessarily dependent upon the storage of flood waters. Quoting Mr. Williams,

"Periods of from five to seven years, in which the flow is insignificant, may be expected, and consequently large carry-over storage is necessary. In order to furnish a sufficient irrigation and municipal supply from Rodriguez Reservoir, a capacity of about 110 000 acre-ft. will be necessary, requiring a maximum reservoir water surface about 180 ft. above stream bed."

It is estimated that a flood of 150 000 sec-ft. is possible.

Description of the Site

The site is described by Mr. Williams, as follows:

"The Rodriguez site is in a gorge, having at its most narrow section, a width, at stream bed, of about 100 ft., and at 130 ft. above stream bed, a width of about 750 ft. Beyond this, on either side, the gorge is flanked by a saddle and by gentle slopes, which, immediately adjacent to the site, rise to 200 ft. or more above stream bed. The saddle on the westerly side is to be utilized in the construction of the spillway.

"Rock outcrops throughout a considerable portion of the site, the rock of the canyon walls at the gorge, that of the western slope, and that of a portion of the eastern slope being rhyolite, while that of the easterly portion of the eastern slope is granite. The rock is fused at the contact between the granite and the rhyolite, showing the former to be the older rock, and indicating that no great leakage from the reservoir may be expected along the contact. The surface rock of the canyon walls, in the most narrow section, is fresh and hard, although considerably broken. That of the slopes is generally badly weathered, and in some portions considerably disintegrated. On either side of the canyon, are a number of cleavage planes, which dip toward the stream bed and, in some cases, up stream."

Preliminary Investigations

Nine borings were made in the stream bed and fifty-two test pits were excavated on the side hills. The foundation material was described by Mr. Williams as follows:

"The test pits indicated that, on the side hills, suitable rock for foundation could be secured, ordinarily, with comparatively shallow excavation. At the site of the east wing of the dam, for a distance of about 500 ft., suitable rock was found at depths ranging from 15 to 25 ft., while at the site of the proposed spillway, on the west side of the river, the depth for about 100 ft. was from 15 to 40 ft. Five of the nine borings in the stream bed indicated satisfactory rock at depths not exceeding 50 ft. From four of the borings, no cores could be obtained, the borings being made easily with a chopping bit. The materials obtained from two of these four were clean sharp stone chips, but the materials from the other two gave considerable evidence of disintegration."

A geologist who first examined the site, according to Mr. Williams, reported that it would be suitable for a concrete dam, except that he considered an earth and rock-fill dam to be best for the saddle on the eastern bank.

Description of the Dam

The details of the Rodriguez Dam may best be described in the words of Mr. Williams. Quoting,

"The dam is of the Ambursen type. Its maximum height will be 187 ft. above stream bed and about 240 ft. above the lowest foundation bed-rock. The length of crest will be about 2 000 ft. For the central portion of the dam and for the west wing, the spacing of buttresses is 22 ft. center to center. The buttresses for the central portion and the west wing vary in thickness from 19 in. at their thinnest section, to 66 in. at 200 ft. below the crest. The thickness of deck varies from 25 in. at 15 ft. below the crest to 64½ in. at its lowest portion, 192 ft. below the crest. The lower face of the deck has a slope of 1 to 1, and the down-stream face of the buttresses a batter of 1 to 5.

"In each of three of the stream-bed bays of the dam, there will be a 5 by 5-ft. sliding sluice-gate, operated by an hydraulic cylinder. The service outlet works will consist of two 30-in. cast-iron pipes, at an elevation about 33 ft. above stream bed, in each of which will be installed one 30 by 24-in. Johnson needle-valve, and two 30-in. emergency gate-valves. The discharge will be measured by means of a Venturi meter.

"The spillway, which will be in the west wing of the dam, will be controlled by nine 30 by 30-ft. structural steel gates on caterpillar bearings. With elevation of water surface 6½ ft. below the crest of the dam, the spillway will have an estimated capacity of 150 000 sec-ft."

Foundation for the Dam

In September, 1929, consultants were called to consider the factors affecting the suitability of the site for the type of dam proposed. "The problems to be considered," said Mr. Williams,

"Were the possibility of excessive leakage from the reservoir, the practicability of obtaining adequate foundation for the dam, the probable depth and extent of cut-off wall required, and the possible danger of future seismic disturbance. The Consulting Geologist, Dr. F. L. Ransome, reported that:

"'The site chosen for the Rodriguez Dam although in most respects satisfactory, has one grave fault; the river at the site follows a fault zone. The fault is apparently not active and has not moved appreciably for 100 years or more. There is no certainty that it may not move in the future although movement is not expected.

* * * * * * * *

"'Although the existence of the fault zone at the dam site unquestionably is objectionable and although the possibility of future movement on the fault cannot be wholly disregarded, I do not consider that the risk is so great as to justify condemnation of the site, on which work is now far advanced. With full realization of the geological conditions the engineers can take such precautions as will make the chances of failure small, although they cannot be entirely avoided.'"

The other Consulting Geologist, Dr. Paul Waitz, reported as follows:

"Although not with absolute surety, it can be stated, in conclusion, that the faults that have been discovered in the river bed are not caused by modern

movements and it is not probable that movements along the faults will occur which would cause damage to the dam." (Translation from the Spanish.)

The photographs show the materials in the main fault zone and in the mid-stream faulted area. In analyzing the findings, Mr. Williams stated that,

"The bearing power of the rock in the stream bed differs greatly in different parts of the foundation area. Four of the buttresses, if founded on the natural material, would rest for a portion of their lengths upon material of relatively poor bearing power. The easterly two of the four would be founded for a portion of their lengths upon the material of the main fault. It was realized, therefore, that it would be necessary to support the four buttresses by a structure of a type such that the loads from the weaker portions of the foundation area would be transferred to those portions having ample supporting power."

The design finally chosen was an Ambursen type of dam especially adapted to a foundation of variable strength. For this purpose, according to Mr. Williams, a voussoir concrete arch was constructed which was continuous from the up-stream cut-off wall to the toe of the dam. The extrados was curved and the base was flat, resting on the natural material. In this way the pressures will be directed toward the most suitable parts of the dam site, relieving the less suitable parts. Thus, said Mr. Williams:

"The arch is designed to carry the full superimposed load. It is expected, however, that a considerable part of the load will be supported through the sub-intradosal concrete by the natural material in the stream bed. Where, however, such material has not supporting power sufficient for the entire load, and there is incipient settlement, a portion of the load will be taken by the arch, thus relieving the natural material of such portion of the load as may be necessary to prevent further settlement."

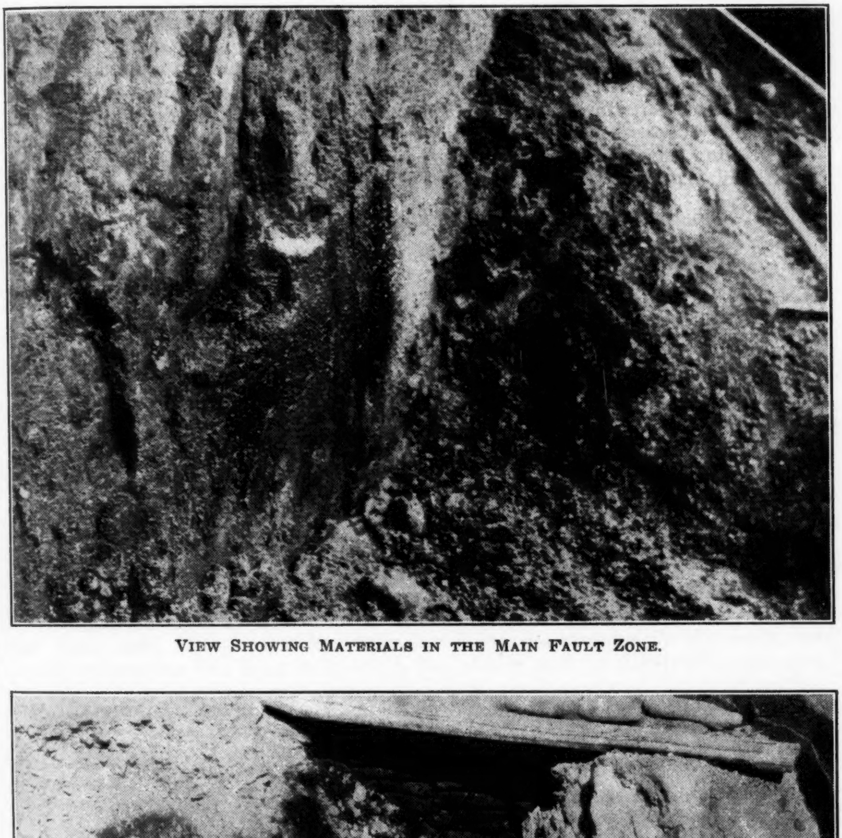
The paper described the elaborate preparation of the foundation before constructing the arch. This involved cut-off trenches across the stream bed, a concrete mat, and key-ways to prevent sliding.

Provisions are being made to study the water pressures that develop under the foundation. Thirty-five 4-in. galvanized pipes leading vertically from the bottom up to observation platforms above high water, will make it possible to read the elevation of the water in the riser pipes.

Mr. Williams described briefly the method of designing the arch barrel. The working loads adopted were: For maximum compression, 700 lb. per sq. in., and for maximum tension, 50 lb. per sq. in. The final designs indicated no tension in any arch section and, therefore, no reinforcement was used.

The design of the transition structure between the flexible arch barrel and the rigid cut-off wall, according to Mr. Williams,

"Consists of a massive block of concrete, having a vertical depth ranging from 40 to 50 ft. and spanning the entire stream bed. The block overlaps the cut-off wall and is tied by steel reinforcement to the arch barrel. The bottom of the block is reinforced longitudinally with three layers of $1\frac{1}{2}$-in. square steel bars, spaced 12 in., center to center, to resist possible tension due to banding in areas where the natural foundation has insufficient supporting power. It is reinforced at its easterly end by seven courses of $1\frac{1}{2}$-in. square, diagonal tension bars, spaced 24 in., center to center, to aid in carrying the load at the fault to the solid rock on the eastern bank. It is also reinforced for possible diagonal tension due to its support by the cut-off wall.



VIEW SHOWING MATERIALS IN THE MAIN FAULT ZONE.

VIEW SHOWING MATERIAL IN THE MID-STREAM FAULTED AREA.

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"In order to prevent dangerous vibration, when water is discharged under high head through the sluice-ways, which are to occupy the three bays above the central portion of the arch barrel, it was considered necessary to surround the gate openings by massive concrete. At the same time it was necessary to provide for flexibility sufficient to permit arch deflection. To accomplish this the massive concrete of the sluice-way installation is to be constructed in blocks, a coating of mastic being used between the blocks and the buttresses, the blocks and the arch, and between the upper and lower blocks."

Cut-Off Wall

Referring to the extension of the cut-off wall to a great depth at the fault line, Mr. Williams stated:

"When the fault in the stream bed was discovered, it was realized that it would be necessary to carry the cut-off wall at the fault to a great depth. There was considerable doubt, however, regarding the depth to which it would be necessary to construct the wall in the western portion of the stream bed. Excavation of the cut-off trench was begun in open cut and was carried by this method to a depth of from 26 to 33 ft. below the original rock surface, corresponding to depths of from 66 to 72 ft. below stream bed. The greater part of the trench required timbering.

"When these depths had been reached, maintaining the open cut had become so difficult that a change of method was necessary and it was decided to continue the excavation through shafts. The trench was filled with concrete, in which were constructed ten shafts through which the excavation and placing of concrete have been continued. The walls of the shafts have been reinforced throughout the zone of inferior rock and wall into the solid rock on either side. Stop-water key-ways are being constructed in the walls, and, when excavation has been completed, the shafts will be filled with concrete."

DISCUSSION

THE GEOLOGY OF THE SITE

By F. C. Finkle,* Esq.

In his discussion, Mr. Finkle outlined the result of examinations of this site made about thirty-five years ago, in which the conclusion of the consulting geologist was that an earth dam would be the proper structure. Reviewing the evidence, and giving to Mr. Williams and his Consultant full credit for precautions taken to avoid structural weaknesses, Mr. Finkle still adhered to the belief that the earth dam would have been the most suitable.

At the time mentioned, the consulting geologist reported that this area lies within a very complex geological zone. The main shear fault was in the Pacific Ocean between the mainland and Coronado Island. The crack existing at the Rodriguez Dam site, according to Mr. Finkle, is one of several that are caused by unequal pressure in the movement along the main shear faults. There is no evidence of movement along this crack in recent times, according to Mr. Finkle, but in the event of such movement in the future, a dam flexible enough to adjust itself would be preferable to one of the more rigid types.

* Los Angeles, Calif.

HYDRO-ELECTRIC POWER DEVELOPMENT AS AN AID TO IRRIGATION

By C. C. Cragin,* M. Am. Soc. C. E.

Francis J. O'Hara, M. Am. Soc. C. E., read the paper by Mr. Cragin entitled, "The Development of Hydro-Electric Power as an Aid to Irrigation, Both in Connection with Storage of Water in the Mountains and Pumping of Water in the Valleys."

In an irrigation project, the availability of an acre-foot of water may mean success to the farmer's agricultural operations; or, on the other hand, the lack of it may mean failure, according to Mr. Cragin. He emphasized the potential danger in developing power from water intended primarily for irrigation purposes. "It must be accepted as a fundamental principle", he said, "that power development in connection with an irrigation project is only justified when there is little or no interference with the irrigation system."

Quoting further,

"The operating organization of a purely irrigation enterprise is rarely adapted to the production and marketing of such a commodity as electrical energy. This is a highly specialized industry and requires the services of a force of specialists entirely foreign to the activities encountered in an irrigation enterprise. To a public utility or a power company operating independently on a fairly large scale (in which case there would be ample assurance of an adequate stand-by service) a development which might reasonably be expected to show a profit of from 8 to 10% on the investment would be considered a favorable undertaking. The same physical conditions and the same development made thereunder by an irrigation project might be a very doubtful investment. A much larger profit on the increment investment should be the minimum set for the same development by the irrigation project.

"Most irrigation enterprises are developed to the maximum extent economically feasible at the moment, with or without provision for future expansion, depending on local conditions. Financing is frequently difficult and the expenditure required per acre of land benefited sometimes approaches the allowable limit. To saddle such an enterprise with the risk of power development likely, during water shortage or other periods of stress, to become a burden to land already heavily taxed, might, if made with only the ordinary margin of profit, easily place the financial affairs of the entire irrigation enterprise in serious jeopardy and even cause many individuals to lose their entire investment."

Another important point mentioned by Mr. Cragin was that an irrigation development may only be considered advisable when the margin of profit assured is greatly in excess of that required in an independent power development. One method suggested by Mr. Cragin was,

"In projects with only a reasonable margin of profit, the element of risk may often be shifted to other shoulders by leasing or selling the power privileges under appropriate contract which will assure freedom from interference with the water supply or operation of the irrigation system, and provide power for project needs under favorable terms."

* Gen. Mgr. and Chf. Engr., Salt River Valley Water Users Assoc., Phœnix, Ariz.

If power rights were found non-salable, this fact in itself should be held as a criterion for discouraging production of power by the irrigation project. From these considerations, Mr. Cragin maintained that, "only under most unusual circumstances will an irrigation project be justified in engaging in the general power business."

Opportunities for hydro-electric development in connection with irrigation projects have been found surprisingly few. According to Mr. Cragin,

"Out of the twenty-eight irrigation projects of the U. S. Bureau of Reclamation now operating, on which there are forty-two storage dams, hydro-electric power has been developed on only twelve. On three of these projects, developments are of small capacity only—less than 200 kv-a.—so that there are really only nine commercial developments."

The Salt River Project faces almost every condition that can arise in connection with a proposed hydro-electric development. Mr. Cragin pointed out that at present it operates eight hydro-electric plants with a combined generating capacity of 103 000 h.p. The project is primarily an irrigation enterprise and serves water directly to 250 000 acres of project land and 90 000 acres of non-project land.

Use of Power

In the opinion of Mr. Cragin, when power is to be developed in connection with irrigation its use and marketability are best adapted to sale in large blocks, especially for irrigation pumping, industrial purposes, or for sale to public utilities that distribute power at retail. Another use of power is in construction work done in connection with the irrigation project. In brief, using the Salt River Project for illustration purposes, Mr. Cragin showed that:

"Benefits to be derived from hydro-electric power installations in connection with irrigation works may range all the way from the use of power for construction and other project purposes to the development of power as a commercial proposition independent of irrigation works, but providing incidental irrigation benefits in the form of better water service and permitting the reclamation of additional land."

The development of the power system on the Salt River Project was summarized by Mr. Cragin, as follows:

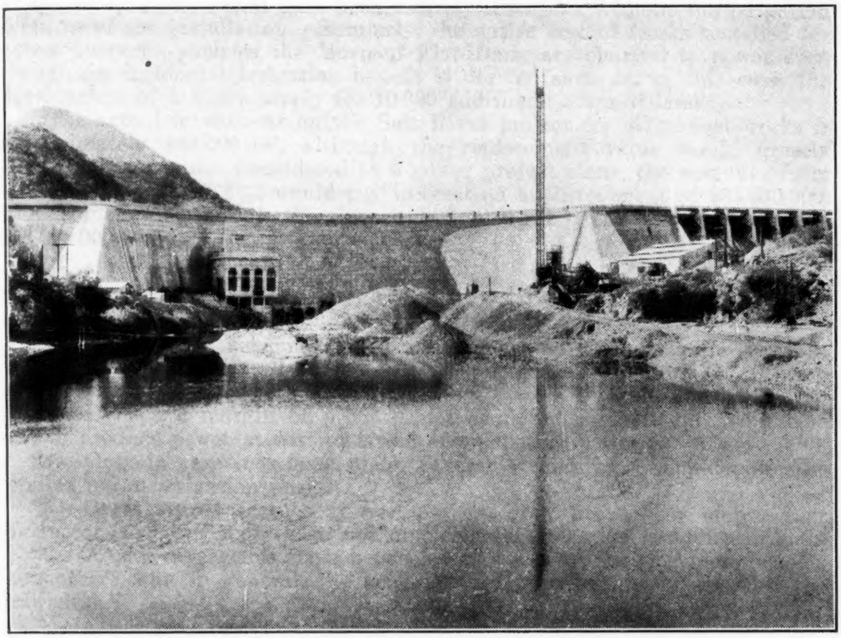
"The first power developed was purely for use in construction. With the completion of the permanent Roosevelt plant, however, capacity was made adequate to supply current in bulk to a local public utility company for general retail distribution. The original South Consolidated Power Plant and the Arizona Falls Power Plant were installed at drops on the main canals, with the Arizona Cross Cut Power Plant, total 10 000 h.p., to meet the growing demand for power from the project system. These developments were financed by direct assessments. The Chandler Power Plant, a 600-kw. installation made in 1919, was financed by the Magma Copper Company, under a contract whereby that company agreed to take the entire output of the plant. In 1920, the combined generating capacity of the Roosevelt plant and the four valley plants was 20 000 h.p. and the investment of the Association in the power system was $4 500 000. The growth of the power demand in the meantime had been such that the Association was faced with the problem of installing additional generating capacity to take care of the market or of leaving the field open for such installations by other competing interests. The Associa-

tion's power business was undoubtedly profitable and its expansion therefore was simply a question of whether or not the Association, as an irrigation enterprise, was disposed to expand its power interests to any degree warranted by the market. An exhaustive investigation and study of all the conditions was completed and embraced in a comprehensive report in February, 1922.

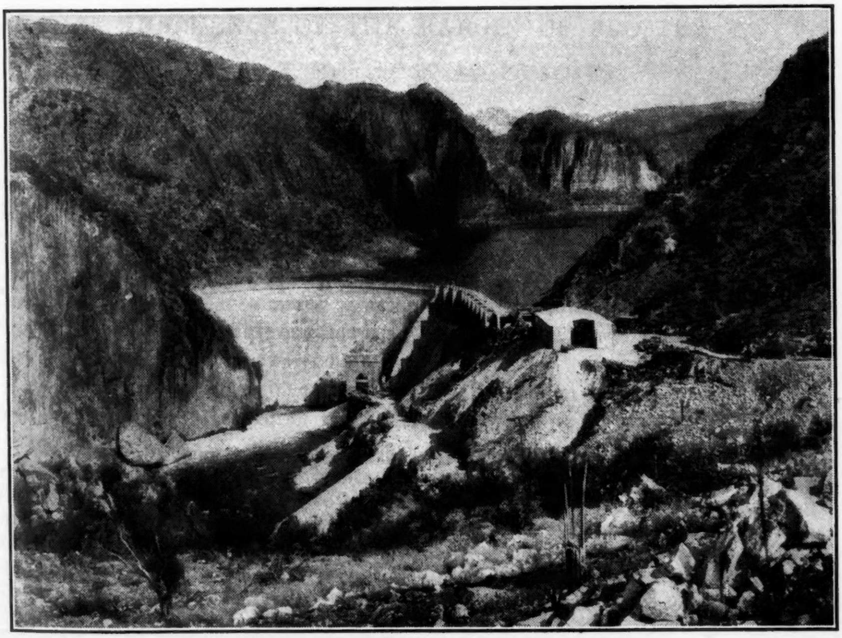
"The total yearly power consumption of the State at that time was 500 000 000 kw-hr., of which 400 000 000 kw-hr. was located within a radius of 100 miles of the project plants; more than one-half of these being in reach of existing transmission lines. Most of this load was generated by steam at costs approximating 1 cent. per kw-hr. It was shown to be possible by installation of 15-ft. gates in the Roosevelt spillways, and by the construction of three dams on the Salt River below Roosevelt, to make available a net head of 729 ft. out of a total of 832 between the top of the gates at Roosevelt and the crest of Granite Reef diversion dam. The increased generating capacity found to be economically feasible was 85 000 h.p., which was susceptible of immediate absorption by the then existing market. The sites along the entire Salt River had been reserved for the benefit of the Salt River Project by the Secretary of the Interior in 1903, but it was obvious that in the event that the Association failed to avail itself of those sites and to develop their potentialities it could not reasonably be expected that they would not eventually be awarded to some one able and willing to develop them. This was the logical source of power to meet the local demand, and since the Association was already in the power business on a fairly large scale, the protection of its existing $4 500 000 investment made the further expansion of this activity a necessity, in addition to its advisability from the standpoint of a profitable undertaking. The construction period extended over seven years, from 1923 to March, 1930. The increased storage facilities provided by the three dams in connection with the 15-ft. gates at Roosevelt, mounted to 648 000 acre-ft., bringing the gross storage on Salt River to 2 015 000 acre-ft. While this additional storage was created solely for power purposes, and while the hydrographic records of Salt River indicated that its development would not have been warranted by the irrigation benefits alone; yet it is nevertheless true that this additional water would be available in unusual periods of drouth, if needed, to increase the irrigation supply for the project lands.

"The main benefit of this development, however, is to be measured in actual dollars and cents. Disregarding the loss which might have been sustained had the Association failed to make this investment, leaving it to be made by competing interests, it is estimated that once the reservoirs have been filled the gross annual power income will exceed $3 500 000, with a net profit of nearly $1 500 000, including power used for project purposes. This income is expected to pay all project indebtedness, all operation and maintenance costs, and to leave a substantial surplus besides. In comparison with the agricultural part of this enterprise, the net anticipated annual power income approximates $7.00 for each acre of land, a cash crop available whether or not the land itself is cultivated. During the past nine years of drouth and during the period of development the net profit has averaged more than $600 000 per year, or $2.62 per acre per year. These profits and estimated future profits are based on the additional costs and investment incident to the production of power and do not include any portion of the original investment in Roosevelt Dam, or the canal system, except actual power structure and equipment.

"In the last analysis the benefits accruing to any irrigation enterprise from power developed in connection therewith could probably be reduced to terms of dollars and cents, even as the benefits from the irrigation development itself. In the case of the Salt River project, the cost of works installed for irrigation purposes and which would have been necessary therefor with or without the power are charged entirely to irrigation, even though the power actually developed in connection with such works, such, for instance, as that



STEWART MOUNTAIN DAM: 17 500 HORSE-POWER PLANT.



MORMON FLAT DAM: 10 000 HORSE-POWER PLANT.

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at Roosevelt Dam, would have been entirely infeasible without the irrigation features of the installation. Similarly, the entire cost of works installed for power purposes, such as the Mormon Flat Dam, are charged to power even though an incidental irrigation benefit is derived such as, in this case, the development of a water supply for 10 000 additional acres of land.

"The actual investment on the Salt River project for all project works is approximately $29 000 000, although the replacement value would greatly exceed this amount. Considered as a power project alone, the revenues from this development, at 6%, would pay interest on an investment of $37 000 000. The actual amount of the investment charged to power, however, is $18 000 000."

Conclusions

Six general conclusions were emphasized by Mr. Cragin, to wit,

"1.—The power development must not interfere with the irrigation project.

"2.—The margin of profit should be much greater than in an ordinary hydro-electric development.

"3.—If such a margin of profit is not available, power rights should be sold to produce power at cost of steam or competing power.

"4.—Only in rare instances, under favorable conditions, should power be retailed by an irrigation project.

"5.—Great benefit accrues to most projects from availability of power at low increment cost for drainage and supplemental irrigation pumping, or both.

"6.—A wide divergence exists between projects, some showing more benefits than others, but, in general, the benefits have overwhelmingly exceeded the drawbacks."

DISCUSSION

INFLUENCE OF THE MARGIN OF PROFIT

By R. V. Meikle,* M. Am. Soc. C. E.

The conclusions reached by Mr. Cragin can, in the opinion of Mr. Meikle, be applied in general to the irrigation districts in the Northern San Joaquin Valley. The total installed capacity on the Stanislaus Tuolumne, and Merced Rivers in this region was given as 120 000 h.p. and the yearly output about 390 000 000 kw-hr.

Mr. Meikle did raise one question, however, and that in respect to Mr. Cragin's assertion that a much greater margin of profit is necessary to the irrigation project in the development of hydro-electric power than in the ordinary development by a power company.

An ideal use of power in connection with irrigation is that created by the need for drainage. According to Mr. Meikle,

"Drainage in some districts has been a problem almost as important as irrigation and the solution of this problem in the Northern San Joaquin Valley, as in the Salt River Project, was found in the operation of deep-well turbine pumps electrically operated. This plan of drainage in connection with irrigation power development has improved the output characteristics of the power-generating plant by providing an ideal power load and by furnishing an irrigation supply which to a certain extent relieves the draft on the storage reservoir."

* Chf. Engr., Turlock Irrig. Dist., Turlock, Calif.

Furthermore, Mr. Meikle stated:

"The power developed by means of irrigation storage may be considered as a by-product if the district depends solely upon this source of power. However, a district may be in a position to provide additional storage and auxiliary hydro-plants with steam, or Diesel stand-by plants and after-bay regulation, with the result that a block of firm power is generated. This full development would make distribution of power by the district possible, or, if sold to the power company operating in the locality, would result in a price to the district two or three times as great as the price paid for seasonal power or dump power. It is probable that the question of power distribution by irrigation districts will not be of great importance in future storage development due partly to the inability of districts to finance such an expensive and difficult undertaking, but principally due to the fact that the power companies are co-operating with the districts to the fullest extent not only in the operation of the present district projects, but in the planning of new ones.

"Power development in Northern San Joaquin Valley in connection with irrigation storage has been in operation for several years under various plans for the disposal of the electrical energy produced."

Mr. Meikle then proceeded to describe the methods of handling the power output of the Stanislaus River Malones Project, the Merced River Exchequer Project, and the Tuolomne River Don Pedro Project.

Statistics, relating to the cost of power in the Turlock District, were given by Mr. Meikle, as follows:

"Average retail revenue per kilowatt-hour, at customer's meter	16.2 mills
Average cost per kilowatt-hour at customer's meter	10.1 "
Average revenue per kilowatt-hour, wholesale	4.27 "
Average cost per kilowatt-hour, wholesale	3.97 "
Kilowatt-hours, at customer's meter per dollar invested	15
Gross revenue per dollar invested	12.1 cents
Investment per kilowatt of installed capacity	$226.00
Average consumption per domestic consumer	1 490 kw-hr."

For the purposes of this tabulation none of the costs of the Don Pedro Project was charged against irrigation.

In concluding his remarks on this subject, Mr. Meikle said,

"As a result of the development of hydro-electric power by the Turlock and Modesto Districts in connection with irrigation storage, these districts now receive their irrigation storage water without cost; the residents of the districts are furnished electrical energy at rates that represented a total saving of $600 000 in 1929; and, in addition to these advantages, there is a yearly net revenue of 3.6% on the total storage and power investment. A large share of the benefits derived from this project are thus passed directly to the people of the districts. Lowering power rates for certain classes of service stimulates power consumption and increases net income under these conditions."

EXAMPLES OF POWER DEVELOPMENT ON IRRIGATION PROJECTS

By George L. Swendsen,* M. Am. Soc. C. E.

Two examples of power administration were cited by Mr. Swendsen. The development of the Bear River Valley in Southeastern Idaho was one of these. Quoting Mr. Swendsen,

* Chf. Engr. and Mgr., Fresno Irrig. Dist., Fresno, Calif.

"It involved a storage of about 400 000 acre-ft. in Bear Lake, and a development of a long interstate river. After a few years of investigation, the entire project was taken over by a power company. That is an example of where the power company did it all. It developed the storage of Bear Lake to the extent contemplated by the Reclamation Service. Then, as things turned out, the people of the Bear River Valley got the full benefit of that storage for nothing; they did not spend a dollar on it."

Another example presented by Mr. Swendsen was in connection with the Snake River Project in Idaho. This river is now reported to be 100% controlled as a result of the whole-hearted co-operation of the Federal Government, the State, the irrigation interests, and the power company.

The extensive pumping necessary in connection with the irrigation development of the southern part of San Joaquin Valley was another example stressed by Mr. Swendsen. The pumping equipment in the Fresno, Alta, and Consolidated Districts was said by him to equal between 9 000 and 10 000 plants on an area of less than 600 000 acres. One important point stressed by Mr. Swendsen was his belief that the State should guard its rights to establish storage reservoirs on the higher sites that are now in many cases being established as National Parks.

GENERAL DISCUSSION

The high cost of fuel affects the feasibility of developing power on an irrigation project, in the opinion of A. H. Markwart, M. Am. Soc. C. E. Quoting,

"The value of irrigation power is the value of the fuel saved in equivalent steam plants. That comes because practically every irrigation power development requires a kilowatt of steam for a kilowatt of hydro, owing to the general rule set that irrigation draft shall govern. * * * If fuel is high, naturally the irrigation development will have a very fine opportunity, because the only possible charge against the hydro would be the installed capacity in the penstocks and perhaps short conduits, if any. The remainder of the development, the dam and reservoir, etc., will all be necessary to the irrigation development, anyway. Consequently, if the capital charge is low on the irrigation power development and the fuel is high, of course, the place for irrigation power is very good. In general, the best system to absorb irrigation power, is a regional system, one that has a relatively high annual load factor. In other words, such systems as are found in the Southern California area. A load factor of perhaps 44% would not be very helpful in the absorption of irrigation power. One of the troubles that irrigation power is going to suffer in this vicinity at the present time, is the presence of enormous quantities of cheap fuel."

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STRUCTURAL DIVISION

APRIL 24, 1930—2:10 P. M. TO 6:00 P. M.

SOUTHERN PACIFIC COMPANY'S SUISUN BAY BRIDGE

By W. H. Kirkbride,* M. Am. Soc. C. E.

This paper describes a double-track railroad bridge project under construction across Suisun Bay, Calif. It is designed to take the place of train-ferry operations across Carquinez Straits between Benicia and Port Costa. In addition to the main bridge, the job includes other steel bridge structures and about 20 miles of railroad track.

The bridging of Suisun Bay or Carquinez Straits has long been considered, said Mr. Kirkbride:

"In the late '80's' there were newspaper discussions of the project and the Army Point-Suisun Point alignment was considered by railroad officers. Later, a prominent bridge company was requested to make studies, and following this, in 1904, during the time of the late E. H. Harriman, the then Chief Engineer, William Hood, prepared plans for a bridge crossing to replace the ferry system.

"Sundry locations were investigated, including the line up stream from the Army Point-Suisun Point alignment, several in the vicinity of Benicia and Port Costa and others as far west as Vallejo Junction.

"A series of test piles were driven across the Bay which showed the bottom to be of soft material, great depth to hard-pan and rock strata, and, of course, no knowledge as to the exact character of the bed-rock so developed.

"It was decided to build a low-level bridge consisting of a swing draw span with a certain number of spans and ballast deck trestle approaches. Application was denied by the War Department and the project was dropped for the time being; subsequently, the bridge studies continued, a crossing near Antioch being considered in connection with the extension of the Walnut Grove line, but this route was rejected."

In 1927, traffic conditions were such that the two existing car-ferries would have to be replaced by three new ones, or a new bridge would have to be built. The latter expedient was found to be the most economical, and a location was chosen between Army Point on the north shore of Suisun Bay and Suisun or Bull's Head Point near Martinez.

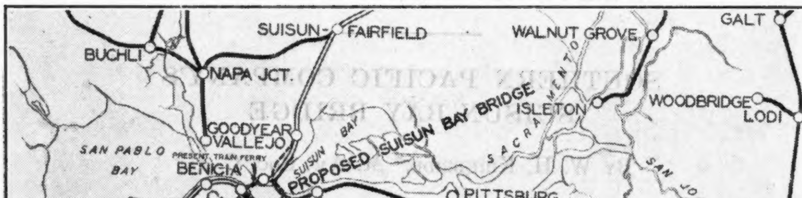
Geologists examining this proposed site, to quote Mr. Kirkbride,

"Reported the existence of two well established major faults extending parallel to each other in a northwesterly and southeasterly direction; one (the 'Southampton') west of the bridge passing under the ferry lanes between

* Engr., M. of W., Southern Pacific Co., San Francisco, Calif.

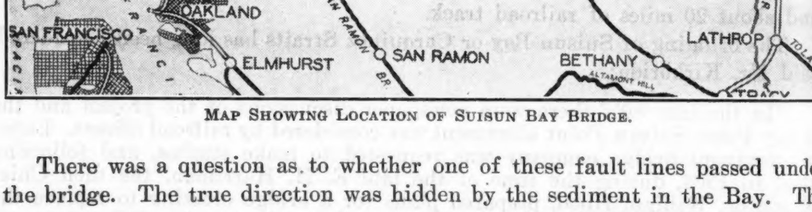
Benicia and Port Costa, and the other (the 'Green Valley') east of the bridge. These fault lines are 5 to 6 miles apart.

"In the Martinez hills there are various fault lines in direction angular to the major faults. The dip of the strata showed a reversal of direction, indicating a folding of the earth's crust, local in extent, and the development of a syncline."



MAP SHOWING LOCATION OF SUISUN BAY BRIDGE.

There was a question as to whether one of these fault lines passed under the bridge. The true direction was hidden by the sediment in the Bay. The geologists differed as to this point, according to Mr. Kirkbride, but all agreed that the fault was not active and that any future movement would be nominal.



SUISUN BAY BRIDGE: CROSS-SECTION OF BAY ON CENTER LINE OF BRIDGE, SHOWING FINAL ELEVATION OF BED-ROCK.

Nevertheless, the bridge piers and superstructure were alike designed to resist earthquake vibration. Accordingly, the piers were planned with their centers of gravity as low as possible. Concrete was specified with a compressive strength of 2 500 lb. per sq. in. and a tensile strength of 400 lb. per sq. in. at the end of 28 days. The steel reinforcement specified was 30 lb. per cu. yd. of concrete, placed so as to withstand accelerations of 5 ft. per sec. per sec. This is twice the intensity of the San Francisco earthquake in 1906 and one and one-half times the intensity of the Japanese earthquake in 1923.

Continuing in the words of Mr. Kirkbride,

"The superstructure was designed to resist similar shocks. The tops of piers were widened and the bases of the steel bridge seats were notched into the concrete to prevent the spans from being pulled off their piers.

"In designing a pier structure of this kind to resist earthquake vibrations, a quick reduction from a large mass to a smaller mass is a source of weakness. Likewise, a construction joint is undesirable; monolithic construction is essential. To secure this, as between caisson and pier shaft, and to overcome the difference of inertia of the two different masses, reinforcing rods extend upward from the outer walls of the caisson, pass through the distributing block, curving inward and upward into the pier shaft."

Construction

The deep-water piers were the most difficult to construct. This work was described by Mr. Kirkbride, as follows:

"The depth of water prevented open coffer-dam work or the use of the pneumatic process—and it was proposed to sink caissons by the open-well dredging method. The depth of water (a maximum tidal fluctuation of about 10 ft.), the swiftness of the current during the winter floods and at rip tide, and the softness of the underlying bay deposits, made the launching and sinking of a timber crib difficult.

"The Contractors, Siems, Helmers, and Schaffner, of St. Paul, Minn., to whom was awarded the foundation contract, in submitting their bid, proposed a very unique method of sinking these deep-water caissons, which is being carried out with great success.

"The accepted construction involved, first, the driving of piles to support an octagonal-shaped platform. Upon this platform eight steel-gallows frames were erected containing winches with cables passing over sheave-wheels set at the top of the tower. By means of this apparatus a steel shell that might be likened to a bottomless oil tank 81 ft. in diameter, was erected and lowered through the water into the mud, which it penetrates to a depth of 15 to 30 ft., depending on the compactness of the material.

"This shell is built up of circular sections, 10 ft. in height, which are bolted together, permitting later removal, except that portion which is in the mud.

"The steel shell rises above high water and is filled in with suitable sand dredged from up the river."

When the caisson has been sunk to bed-rock so that most of the overlying material has been removed, a deep-sea diver is sent down to explore the actual conditions. He reports his findings by telephone. Quoting Mr. Kirkbride,

"In all cases considerable clay or gravel, as the case may be, has been found compacted under the partition walls and banked up against the cutting-edge. By means of the information thus furnished the Contractor is enabled to direct his jetting operations until by successive examinations the foundation area is reported satisfactory and in a clean condition.

"The diver has been able to render accurate information as to the penetration of the cutting-edge into the foundation rock and in each case brings up samples of the rock encountered. We believe this is the first time a deep-sea diver has been used at such great depth of water in open caisson work. It has enabled the railroad engineers to verify the diamond drill borings and to know definitely the nature of the bed-rock formation and to be assured that the bridge piers are actually resting directly upon the rock, that full bearing has been secured, and an interlocked connection established between the concrete, the steel cutting-edge, and the bed-rock."

The concrete work that followed was done with painstaking care, according to Mr. Kirkbride's description of these operations. The concrete seal was poured through a 12-in. tremie pipe, sometimes as long as 150 ft. Tests showed that the concrete in place was of excellent quality.

Of the completed pier, Mr. Kirkbridge states,

"In all cases the piers have landed on bed-rock with but slight variation from the true center and inclination from the vertical. For instance, the caisson of the deepest pier, No. 13, is off from true orientation by a southerly departure of 0.73 ft. and an easterly departure of 0.89 ft., the inclination from the vertical being 0.75 ft. in a height of 114.1 ft.

"Of course, even this slight error is not carried upward, as forms of the pier shaft are set to instrumental exactness.

"Pier No. 12 may be used to illustrate the most accurate condition of placing. In this case the caisson departed from true orientation to the amount of 0.2 ft. northerly and 0.3 ft. westerly; the inclination from true vertical being 0.14 ft. in 86.4 ft., equivalent to a batter of 1 in 600."

Superstructure

The length of the bridge between abutments was stated by Mr. Kirkbride to be 5 603.5 ft. Of this distance, 4 050 ft. across the Bay is navigable, with a clearance above mean higher high water of 70 ft. One 328-ft. vertical lift span provides a clearance of 135 ft. above mean higher high water.

The main characteristics of the superstructure, according to Mr. Kirkbride, are as follows:

"The superstructure consists of seven through truss spans of 526 ft. length (the distance between piers being 531 ft.), one vertical lift span, 328 ft. long; one deck span of 504 ft. length, a deck span, 264 ft. in length, and a viaduct approach on the south end of five 8-ft. girders and four 40-ft. tower girders; on the north end a viaduct approach of one 100-ft. girder and three 40-ft. tower girders.

"The through and deck spans are simple Warren trusses; the lift span is of Warren truss design.

"Truss span lengths of 526 ft. do not represent an economical design, balancing cost of superstructure against cost of substructure. The economical span of length is somewhere between 400 and 425 ft., but navigation interests are better served by a span of 526 ft. primarily to permit the passage of long barge tows without 'side-swiping' of piers. After conference with the engineers of the War Department and navigation interests, the 526-ft. length was agreed upon."

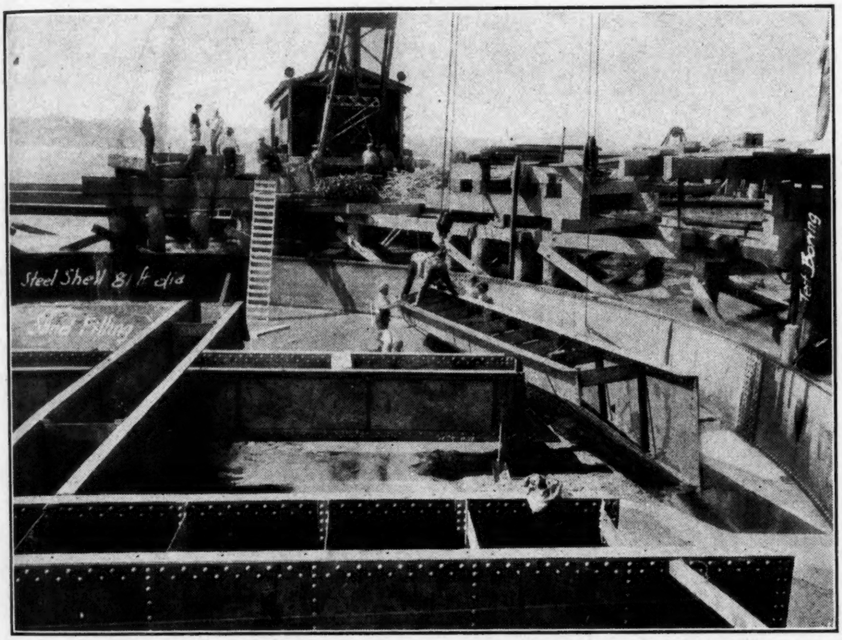
Design

Considerations affecting the design were given in detail by Mr. Kirkbride. Important abstracts of these items are as follows:

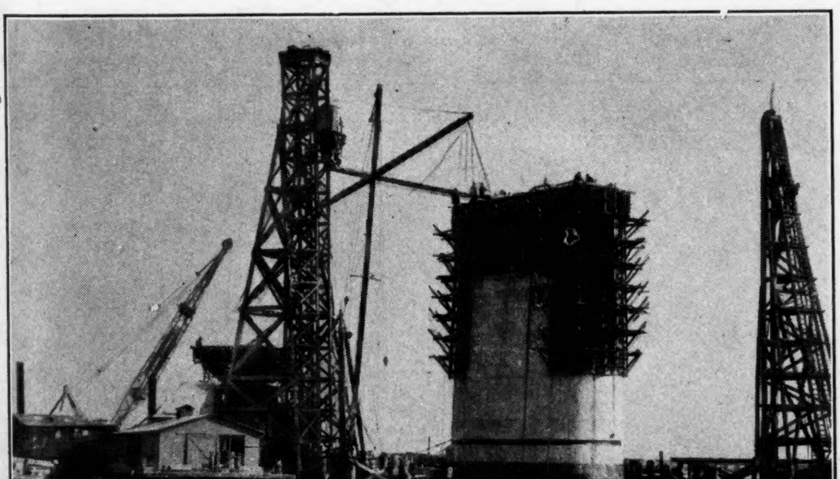
"The superstructure is designed for a train loading equivalent to Cooper's *E*-90 loading, followed and preceded by a uniform load of 7 500 lb. per ft. of track. To give the maximum stresses permitted, the uniform load is omitted or used at both or either ends of the concentrated system.

"For maximum stresses caused by the live load and by centrifugal forces, both tracks are assumed to be simultaneously loaded, except that 90% of the live load is used. This represents 1.5 times the loads now actually running on the track.

"Silicon steel will be used for the principal members fabricated from shapes, and carbon steel for the remainder, except that the tension bars in the lower



SUISUN BAY BRIDGE: ERECTION OF CUTTING-EDGE OF CAISSON, PIER NO. 12.



FORMS SET TO FINAL HEIGHT, PIER NO. 12, SUISUN BAY BRIDGE.

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chord, consisting of I-bars, will be heat-treated. The total weight of the structural steel is 25 000 000 lb. of silicon steel, 5 500 000 lb. of heat-treated I-bars, and 13 500 000 lb. of carbon steel.

"The permissible unit stresses as used in the design are as follows:

"For axial tension in net section:

Carbon steel.....21 300 lb. per sq. in.
Silicon steel.....32 000 lb. per sq. in.
Heat-treated eye-bars.....36 000 lb. per sq. in.

"For axial compression in gross section:

Carbon steel......$20\,000 - 70\frac{l}{r}$ (up to 16 500 lb. per sq. in.)

Silicon steel......$30\,000 - 105\frac{l}{r}$ (up to 24 750 lb. per sq. in.)."

Various methods of erection were considered. It was not thought possible to use timber piling successfully and economically because of the depth of water, the soft bottom, the swift current, and the height of the piers above water. Describing the erection procedure in detail, Mr. Kirkbride explained,

"Proceeding from the abutment, the tower posts and girders were placed by erecting cranes, except that temporarily the last pair of girders of the eastward track was used for a turnout approach to a falsework trestle that was driven immediately adjacent to and up stream above Pier No. 10.

"Then followed the erection of the 260-ft. deck span upon this falsework, which was so driven as to provide for floating two 40 by 125-ft. barges into position under the span. By this means the span was floated into its true position and placed upon Piers Nos. 10 and 11. After this, the 504-ft. deck span was erected upon the falsework. This span is to be used as falsework eight successive times in the erection of the remaining spans of the bridge. This is being accomplished in the following manner:

"After erection the span is floated from its falsework by means of the two barges into position between Piers Nos. 11 and 12 where it rests upon two temporary steel bents that extend to and rest upon the shelf that is formed by the top of the caisson at Elevation —20. These bents are secured to the pier shaft."

In conclusion, Mr. Kirkbride emphasized three features described in the paper as follows:

"1.—Intensive examination into earthquake fault condition and the designing of the bridge piers to be resistant to earthquake shock;

"2.—The creation of artificial islands of sand within steel shells by means of which the all-concrete caisson bridge piers were formed above water and sunk within the steel shells to bed-rock; and

"3.—The temporary use of one of the permanent deck spans as falsework in the construction of the bridge spans across deep water."

DISCUSSION

MISCELLANEOUS CONSTRUCTION DETAILS

By N. F. Helmers,* Esq.

Each new job of any consequence from an engineering standpoint adds some new "wrinkles" to the fund of knowledge. On the Suisun Bay Bridge,

* Siems-Helmers, St. Paul, Minn.

a system of signals used in connection with the triangulation work was mentioned by Mr. Helmers in discussing the paper by Mr. Kirkbride.

Quoting Mr. Helmers,

"The base line was established approximately parallel to the bridge and one-quarter mile down stream from the bridge site. From the two observation stations, all locations are given not only for the falsework, but for the sinking of caissons, erecting of shaft forms, setting of anchor-bolts, etc.

"In connection with pile-driving, signals to the driver are made by the use of a diamond-shaped target revolving about a center. One-half of the diamond is white and the other red. The signaled direction to the pile-driver is indicated by the white half of the diamond target. Under ordinary visibility it can be plainly seen without glasses from any pier. For driving falsework, the pile-driver is spotted at the intersection of two lines sighted from the two observation stations, the direction having been previously calculated to form such intersection for each pile as designated on the contractor's pile plan. To obtain the necessary degree of accuracy in the turning of angles, each transitman turns from his foresight on the movable disk which is set to take care of the odd minutes and seconds of the arc. The vernier is not used. This method gives the location within a probable error of 1 in. The co-operation between the instrument party and pile-driver crew has been so good that we have a number of times driven 20 piles in 2 hours under these conditions. Because of the swift current it is necessary then to stop and cap the piles driven to prevent their being broken off by excessive and continued vibration."

Construction Equipment.—A heavy floating pile-driver, with 90-ft. leads and a 13 200-lb. double-acting hammer, was used on this work. This rig, according to Mr. Helmers, has a revolving derrick with a 75-ft. boom as well as an unusual number of engine drums and winches. The piles driven were from 100 ft. to 125 ft. long, with butts 24 in. to 26 in. in diameter.

A 2-yd. floating concrete plant with a 104-ft. tower is an important part of the equipment. Its normal capacity is 60 cu. yd. per hour. Besides these, said Mr. Helmers,

"There are two additional, heavy, floating derricks; five stiff-leg derricks erected on the falsework of each main pier; five tugs; locomotive crane; pump barge; and a very complete machine shop, besides a number of deck lighters. The cement is handled in two heavy barges, housed and provided with an inclined belt conveyor, power-driven, which delivers the cement on the platform back of the mixer on the concrete plant barge.

"Practically all the machinery used is operated by steam, with oil as fuel. The tugs are operated by Diesel engines. All the plant used on this job is the property of the contractor.

"*Pile Driving.*—At each of the eight main piers, as soon as the first falsework piles were driven, a carefully detailed report of its behavior was wired to the Consulting Engineer, M. F. Clements, M. Am. Soc. C. E., and from the study made by his office, the depth to which the steel shell would sink was estimated and recommendation made for this item, as well as passing on whether or not the mud should be removed from the inside of the shell before placing sand filling. These studies and recommendations have been wonderfully accurate in forecasting the behavior of each caisson and have been of great value.

"One of the important items in handling this job was the behavior of the mud of the Bay bottom. It has been demonstrated that ordinary river silt

gives sufficient bearing to be entirely satisfactory for the method used, only a small amount of settlement taking place under such conditions and this settlement quite uniform. This condition obtained at Piers Nos. 13 to 17, inclusive, and is expected also to obtain at Pier No. 18.

"The driving of falsework piles at Pier No. 12 showed a very soft bottom, the 100 to 110-ft. pile sinking 43 ft. under weight of hammer and pile, and going over 5 ft. under the last ten blows. Both shell and caisson showed some movement under these conditions, and when the dredging in the caisson had gone through the sand filling and relieved the mud under the cylinder of sand, both steel shell and caisson listed, the caisson at one time being 18 in. out of plumb and the shell nearly 8 ft. At certain times the dredging would be 5 ft. below the bottom of the dredging wells on the high side (all that was permitted), and as much as 12 ft. above the bottom of the cutting-edge of the low side. The caisson was landed within 6 in. of the correct theoretical position.

"Pier No. 19 has the softest ground of all. The material was described on the original Southern Pacific drawing as 'blue mud with vegetation' and the falsework piles used were from 120 ft. to 126 ft. long, extending practically to the rock. It is expected to use from 100 ft. to 110 ft. of shell in the sand island for this pier and of this the three remaining pieces of the tenth 10-ft. ring are now being placed. The weight of the sand island is pushing the very soft material out from under it, moving a portion of the falsework upward with the flow. Instead of at all times settling uniformly the shell and its sand filling at times takes its settlement by jerks; sinking 3 or 4 ft. on one side and then leveling itself in the next few hours. On April 22, 1930, this shell was 7 ft. out of level.

"An attempt to cure this by loading with sand and jetting on the high side was practically unsuccessful, but by dredging for 24 hours on the high side it was brought to within 1 ft. of level, when the dredging was stopped. More sand will be added to get it still nearer level. The intent of the procedure at this time is to obtain most of the settlement if not all, before the caisson or pier proper is started.

"An interesting detail of the method used is that the maximum weight is secured at the start; this being when the sand-fill is completed and 25 ft. of caisson added. From that point the weight of the sand removed and the water displaced by the concrete outbalance the progressively increased weight of the concrete caisson.

"Under this method of sinking, the sand between the steel shell and the caisson can always be removed, thus relieving skin friction and giving the greatest weight desired to permit rapid sinking. Under normal conditions a caisson can be sunk at the rate of 10 ft. in a 4-day cycle, and at times this has been cut to 3 days."

COMMENTS ON THE SUPERSTRUCTURE

By E. J. Schneider,* M. Am. Soc. C. E.

As a part of the preliminary studies of this project more than twelve separate estimates were made of the steel required for various combinations of designs. For example, Mr. Schneider pointed out that the steel for single-track, double-track, and gauntlet-track bridges was computed. With these

* Contr. Mgr., U. S. Steel Products Co., Bridge and Structural Dept., San Francisco, Calif.

combinations, the effect of providing 20-ft. clearance instead of 70 ft., was studied. Quoting Mr. Schneider,

"It was considered as to whether a highway should be built in connection with the bridge. Estimates were made with the highway and without the highway, and also providing for the highway at some future time. Furthermore, carbon, heat-treated, and silicon steel were considered; also continuous, cantilever, and simple spans."

Finally, a through-type Warren truss was chosen, to be designed for two tracks under an *E*-90 loading followed by a uniform load of 7 500 lb. per ft. Under these conditions, the total weight was computed as 25 000 000 lb. of silicon steel and 5 500 000 lb. of heat-treated eye-bars.

Mr. Schneider's comments on testing are interesting:

"While it is customary for us to test to destruction, one or two eye-bars from every heat, the question was asked, 'How do we know that the ones that are not tested are all right?' This was given much consideration on account of the size of the bridge and the amount of money involved, and it was finally decided that each bar should be proof-tested. As a result the 1 807 bars were tested in the full-sized testing machine at Ambridge, Pa., and every bar came well within the required limit. This confirms the opinion of the engineers of the fabricators who have always maintained that the eye-bar is one of the most dependable sections in a structure.

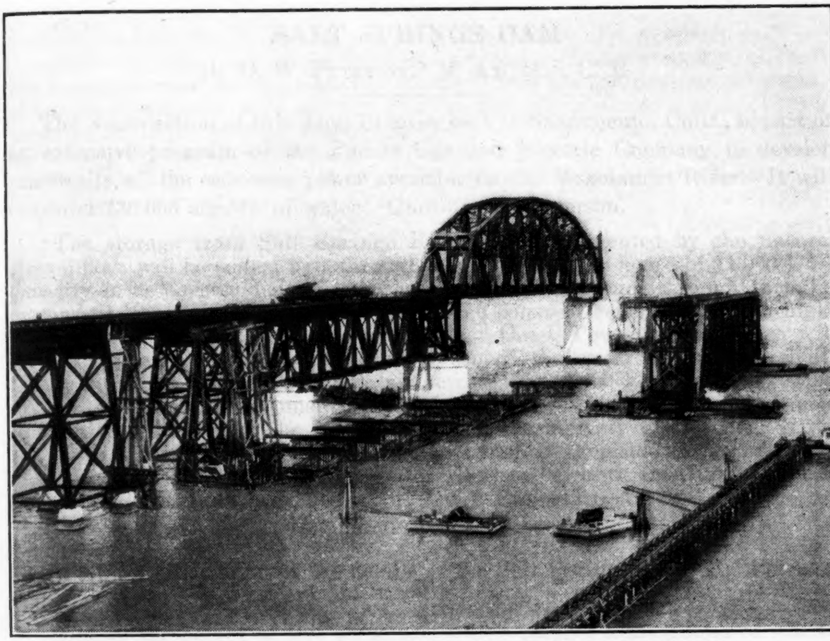
* * * * * * * *

"When the preliminary studies were made, it was first thought that a span of approximately 408 ft. would be an economical length, but the economical curve was about horizontal between 400 and 550 ft. and this was settled when the War Department ruled that the distance between center to center of piers should be 530 ft. This decision, however, was not reached until the Army engineers had made several tests of long tows in the vicinity of the bridge where swift tides and cross-currents frequently occur. These experiments indicated that a longer span was necessary to provide ample clearance and to prevent long tows from being 'side-swiped.' The War Department ruled on a 70-ft. underclearance from mean higher high water for all except the lift span. This means that all river crafts can pass under the bridge and only sea-going vessels will be required to use the Government channel where a lift span is provided with 300 ft. horizontal clearance and 125 ft. vertical clearance above mean higher high water."

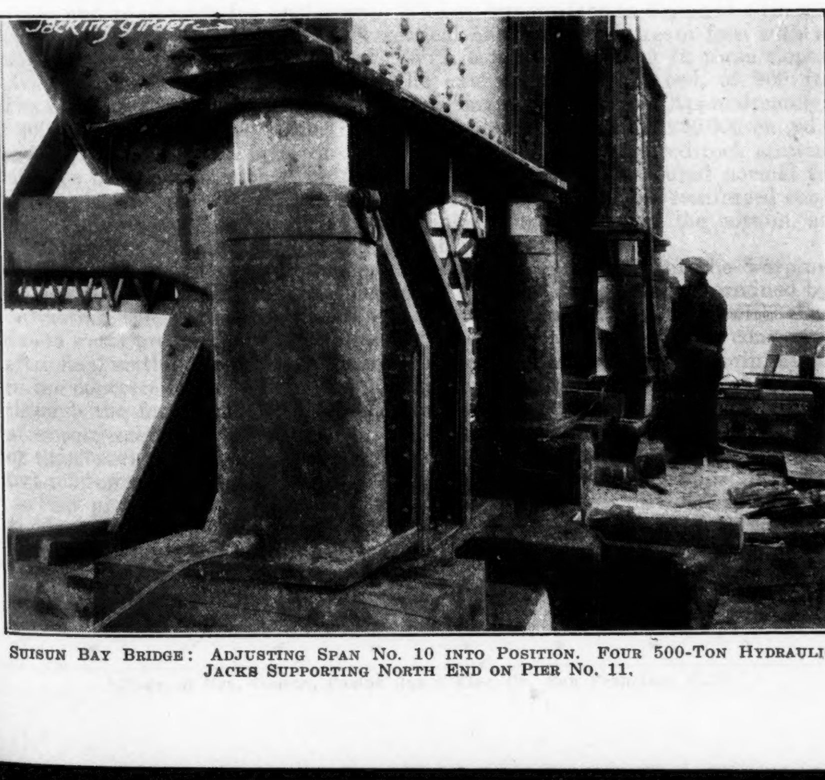
In the erection work,

"Barges used for floating operations were those originally designed and built for the erection of the Carquinez Highway Bridge located about seven miles down stream from this present structure. They are 40 by 130 ft. with 10-ft. depth, and have a capacity of 1 000 tons. When used on the Carquinez Bridge the maximum load they carried was 800 tons. When loaded to 1 000 tons capacity they had a free-board of 1 ft. 6 in. and extra waling strips were provided on the side so that the deck would be kept dry in case of rough weather.

"The first span was floated on February 28. Advantage was taken of the low tide when the barges were placed under the span. Shortly after noon, the floating was completed, and the span was resting on the temporary bents between the piers. The bents supporting this span were of sufficient length so that the span when erected on this falsework would be about 3 ft. above its proper elevation. Hydraulic jacks were used to take care of any adjustment which might be necessary."



FLOATING SPAN NO. 19, OPPOSITE SPAN NO. 11, SUISUN BAY BRIDGE.



SUISUN BAY BRIDGE: ADJUSTING SPAN NO. 10 INTO POSITION. FOUR 500-TON HYDRAULIC JACKS SUPPORTING NORTH END ON PIER NO. 11.

SALT SPRINGS DAM

By O. W. Peterson,* M. Am. Soc. C. E.

The construction of this dam, 70 miles east of Sacramento, Calif., is part of an extensive program of the Pacific Gas and Electric Company to develop practically all the economic power available on the Mokelumne River. It will impound 130 000 acre-ft. of water. Quoting Mr. Peterson,

"The storage from Salt Springs Reservoir supplemented by the natural stream flow, will be passed first through a low-head power house of 11 000 k-va. capacity to be constructed near the base of the dam, thence through 18 miles of conduit, consisting primarily of reinforced concrete bench flume, to a high-head (1 220-ft.) plant to be known as Tiger Creek Power House which will have a capacity of 60 000 k-va. Final use of the water through a rebuilt 'Electra' project will result in a total increase of 146 000 k-va. to the existing system. Ultimate development of additional storage on Bear River, the most productive tributary of the Mokelumne, and the delivery of the Bear River supply to a high-head impulse unit to be installed alongside the 300-ft. head reaction turbine at Salt Springs Power House will result in a total increase of 171 000 k-va. from the Mokelumne River Project."

Design Features of Salt Springs Dam

In a brief description of the details of the Salt Springs Dam, Mr. Peterson stated,

"Due to the height of this structure, which reaches a maximum of 328 ft. above the foundation, great care was used in determining conservative side slopes and to provide for settlement. The up-stream face is designed with an average slope of 1.3 horizontal to 1 vertical and the down-stream face with a slope of 1.4 horizontal to 1 vertical. With a top width of 15 ft. these slopes give a base width, measured along the axis of the stream bed, of 900 ft. The crest at Elevation 3 958.5 has a total length of 1 300 ft. Approximately, 2 900 000 cu. yd. of rock will be required for the dam of which 220 000 cu. yd., or 7.5%, will go into a layer of derrick-placed rock. This placed rock section is to be constructed with a uniform thickness of 15 ft. measured normal to the up-stream face. It, in turn, will be covered with a flexible reinforced concrete facing 1 ft. thick at the top and increasing to 3 ft. at the bottom, at which point it ties into the concrete cut-off wall.

"In plan, the dam is arched up stream; the magnitude of the warping (which is 6½ ft. at the crest and decreases with the depth) was determined by estimating the settlement due to the weight of the fill combined with that due to water pressure, and providing that the dam is to retain a slight convexity after final settlement has taken place. Tension cracks will thus be minimized in the concrete facing. In addition, the design is such that a vertical section through the face of the dam parallel to the stream bed will remain concave after settlement has taken place. Hence the lateral thrust due to the weight of the concrete will be against the rock backing at all times and the tendency to buckling which would result if the section became convex is avoided.

"To give sufficient flexibility, the concrete facing is divided into independent squares approximately 60 ft. on a side. The edges of these slabs rest on concrete supports formed by filling grooves, provided in the face of the placed rock, with concrete. To simplify construction, the sides of the squares are straight lines, the four corners being accurately set, using computed effects from the theoretic slope line.

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* Engr. of Gen. Constr., Pacific Gas & Elec. Co., San Francisco, Calif.

"Two layers of 1-in. round steel bars at 9-in. centers are used to reinforce the lowest portion of the facing; the amount of reinforcing is gradually reduced to a single layer of 1-in. square bars at the crest of the dam spaced 1 ft. 1 in. in both directions. The joints between panels are made water-tight by means of soft copper seals embedded in the concrete. A concrete mix designed for maximum density using 5 sacks of cement per cu. yd., gives a 28-day strength of approximately 3 000 lb. per sq. in.

"The cut-off is a concrete wall 6 ft. thick extending from the base of the dam 6 to 20 ft. into good rock. Where the height of the dam exceeds 200 ft., grout holes, 2 in. in diameter, 50 ft. deep, and spaced 6 ft. apart, were drilled along the bottom of the cut-off trench. At the two banks, where the height of the dam is less than 200 ft. the grout holes are spaced at 10-ft. centers and the depth is gradually reduced to a minimum of 25 ft. at the top of the dam.

"The bottom 10 ft. along the entire base of the derrick-placed rock section is laid up in concrete mortar to give a more gradual transition from the solid bed-rock to the loose fill in order to minimize the tendency of the concrete slab to crack along the line of junction of the facing and the cut-off wall."

A temporary by-pass tunnel, 22 ft. in diameter, was driven under the north abutment of the dam. In his paper, Mr. Peterson explained that this will later be used as a permanent outlet. He described the spillway as a side channel type, excavated in solid rock. In detail:

"The overflow crest, 650 ft. long and 15½ ft. in elevation below the crest of the concrete facing, provides for a 48 000 sec-ft. flood with an 8-ft. free-board. This corresponds to a run-off of 300 sec-ft. per sq. mile over the 160 sq. miles of drainage area, of which the water surface of the reservoir amounts to about 900 acres. The maximum recorded flood, occurring in March, 1928, averaged 12 500 sec-ft. over a 24-hour period, with an estimated peak of 14 500 sec-ft."

This project will entail the handling of 20 000 tons of freight by truck from the Railroad Terminal at Martell, Calif. According to Mr. Peterson, motor-truck transportation, including the building of about 30 miles of construction road, was found to be more economical than the construction of a railroad line for this purpose.

Shops and Camps

Working conditions are described by Mr. Peterson as follows:

"Shops and a camp of simple frame construction for 500 men, including a well equipped field hospital, are located on the left bank of the river, down stream from the dam. Lumber for these buildings was cut by a saw-mill set up near the site, use being made of logs salvaged from the reservoir area. The water supply is from springs, supplemented by pumping from the river. Waste water from the kitchen and sewage is piped to a septic tank from which it flows to a sprinkler system for aeration and filtration before returning to the river bed.

"Electric power is used almost exclusively for construction operations and for kitchen service, the total transformer capacity at the dam being 4 000 k-va.

"The Company furnishes board and sleeping quarters, not including blankets, at $1.25 per day. Common laborers are paid $4.00; carpenters, $6.40, and mechanics, $6.00 to $8.00, per 8-hour day. Work proceeds on a basis of two shifts of 8½ hours each, with every other Sunday a day of rest."

Stripping of Dam Site

"The entire area of the dam site, except for a section of compact boulders in the down-stream half of the stream channel, was stripped to bed-rock, the total amount of material excavated being 312 000 cu. yd. This stripping was

done by power shovels and trucks, the waste being hauled as far as ¾ mile to obtain necessary dump space.

"* * * As soon as an area of the river bottom was entirely clear of débris, boulders and fragments of broken rock were segregated and hauled directly into the section to be built of loose fill, the total amount of rock salvaged in this way being 67 000 cu. yd.

"Much of the débris in the river bottom was so compact that it was necessay to loosen it by powder before it could be excavated. Ten months were required to complete the stripping of the entire site."

Quarry Operation

"Three main quarries at different levels near the north abutment of the dam and excavation from the spillway cut at the south bank furnish most of the rock for the fill. The largest of the quarries, located at an elevation about midway between stream bed and the crest, will provide 1 300 000 cu. yd. of fill material. At the end of 1929 about 1 500 000 cu. yd. of rock had been placed at an average of 125 000 cu. yd. per month for the greater part of the year; it is expected that this average monthly rate will be continued until the latter part of 1930 when limited working space at the upper levels will slow down progress. The present loading equipment consists of two 4-yd., one 2-yd., and one 2½-yd. electric shovels, all working on two 8½-hour shifts for an average of 27 days per month.

"Drilling in the quarries for large shots is done by electrically operated churn drill rigs, which put down holes 5¾ in. in diameter, 60 to 180 ft. in depth. These vertical holes are drilled on 18 to 22½-ft. centers, the successive rows being placed 30 to 45 ft. back from the quarry face. At the base of the quarry face horizontal toe holes, 2 in. in diameter, are drilled with Leyner drills at 8-ft. centers to a depth of about 24 ft.

"On November 5, 1929, a total of 44 churn drill holes, averaging 158 ft. depth, and 99 toe holes of 24 ft. depth were shot with 116 250 lb. of powder, breaking down a total of 231 700 cu. yd. of granite, solid measure, with an average of 1½ lb. of powder per cu. yd. of rock. As much of this material breaks in enormous blocks, secondary drilling with jack-hammers and further use of powder are required to reduce the rock to sizes which the shovels can handle."

Loading and Transportation

"Two full revolving 4-yd. electric shovels mounted on crawler tracks are in use on the north side of the river to load into 30-cu. yd., standard gauge, drop-door, steel dump cars. These shovels load an average of 45 000 cu. yd. per month per shovel, not including about 5% of waste consisting of soil and partly decomposed surface rock. Rock is loaded in all quarries without the use of slings, about 50% being hauled from the dipper teeth directly into cars or trailers without passing through the bucket.

"Storage battery locomotives of 20-ton weight, equipped with compressors for dumping of cars, are used to haul 1 or 2-car trains over a track of 70-lb. rail laid on hewn ties. The four locomotives used were converted from eight 8-ton storage battery locomotives of 3-ft. gauge formerly used on tunnel construction, two of the smaller units being combined and rebuilt to form one of the large locomotives."

Construction of Rock Fill

"To make it possible to start the placed rock at an early date a 75-ft. lift was first built at the up-stream toe of the loose-fill section. Material for this fill and for the lower courses of placed rock was obtained from a small quarry opened for this purpose on the left bank up stream from the dam site. The extreme vertical height of loose fill under which it is considered safe for men to work is 75 ft. For this reason the dam was constructed in lifts of this maximum height.

"The track from the lowest level main quarry running on a — 1% grade was extended out from the abutment in the form of a semi-circle. As soon as a fill of minimum working width was completed across the canyon rock deliveries from this level started at the up-stream slope for the derrick-placed section and at the lower side to widen the fill to its full slope lines. Since the angle of repose is about 1.35 horizontal to 1 vertical, berms will be left at the top of each lift extending beyond the specified slope of 1.4 to 1 on the downstream face. No attempt will be made to obtain a smooth surface; it is expected that rock dumped from the upper slopes will obscure the steps formed by the berms.

"The semi-circular plan of carrying a fill across the canyon is not practical at the higher levels of the dam on account of the limited width. An excellent solution was found in the development of a 48-ton capacity standard gauge steel car which is end-dumped by a power hoist. The application of a power-operated end-dump body on railroad tracks is believed to be new. These cars are proving very serviceable in building narrow end-dump fills without the use of a trestle. The dropping of such large rock would make it extremely difficult to maintain a high trestle and would result in a very unsafe condition for the men on the placed-rock section below.

"Rock in the fill varies in size from quarry fines to 25 tons, one-half the volume consisting of rock more than 3 tons in weight. The average length of haul from the quarries to the fill is about 1 500 ft. Water is used liberally on the dam to wash the fines into the voids and to assist in settlement. Five pump units, each of 1 200 gal. per min. capacity, are used for this purpose.

"In order to dispose of the layer of fine material which accumulates at the top of each lift, shallow pits at about 6-ft. centers are dug over the entire area. Small quantities of low-strength powder placed at the base of these pits are shot, causing the fine material to be loosened and to a large extent scattered. Nozzling of the surface after shooting carries most of the fines into the rock below, leaving a practically clean roughened area for the next fill to bond to.

"Excavation from the spillway totaling about 500 000 cu. yd. is also being used for fill in the dam. Here, two electric shovels, of 2 and 2½-yd. size, load into end-dump, tractor-drawn trailers of 20-ton capacity mounted on caterpillar tracks. The bodies of the trailers are of a special design developed for this job. These units are well suited for this rugged service and for the great amount of turning required in the short haul from the spillway cut."

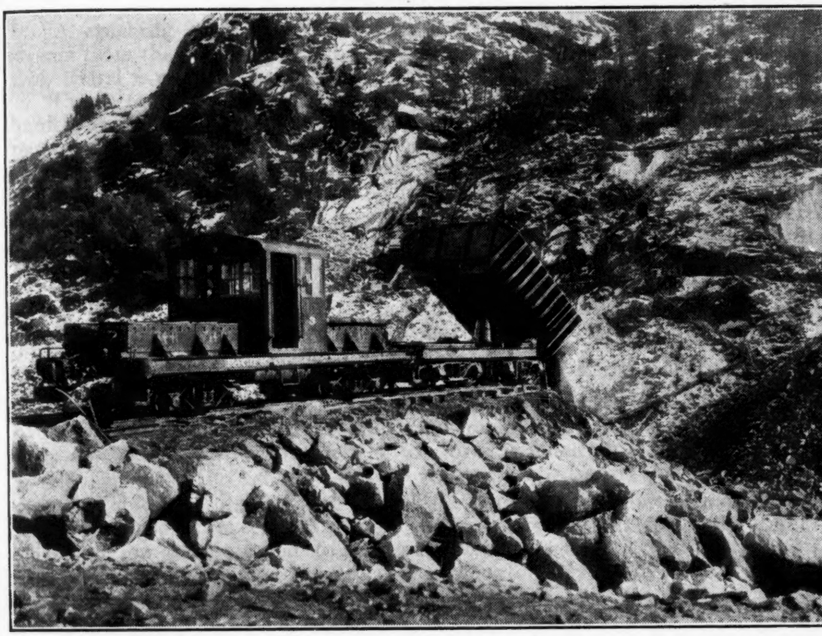
Placed Rock Section

"Building the 15-ft. layer of derrick-placed material containing a total of 220 000 cu. yd., consists essentially of laying down large rock with the best possible contact to the adjacent rocks and of filling the intervening spaces with spalls and small stones. Electrically operated cranes, converted from full-swing power shovels mounted on crawler tracks with 35 to 40-ft. booms and a front drum attachment, transfer rock from the loose fill to the placed section. Rock up to the maximum size of 10 tons is handled by means of wire-rope slings.

"Each crane unit moves ahead a distance of 200 to 300 ft. on a roughly leveled roadway on the top of the course being laid. On its return trip a second course is laid above the roadway using spalls removed from the road bed to chink up spaces in the placed rock. Each of these courses is full width and 6 ft. in depth, making a 12-ft. lift constructed from each road level.

"Each crane, working two shifts with a crew of six men, places approximately 3 000 cu. yd. per month, 15 000 cu. yd. per month for the five units to be used ultimately. Two units only were used in the beginning, increasing in number as the total length of roadway became greater.

"An interval of several months elapses between the dumping of the loose fill and the building of the placed rock against its slope."



END DUMP CAR UNLOADING.

VIEW OF UP-STREAM FACE OF SALT SPRINGS DAM, SHOWING CONCRETE FACING BEING PLACED WITH SLIDING STEEL FORMS, WITH WATER-BALLAST TANKS.

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Concrete Facing

"A crushing and concreting plant was constructed on the left bank up stream from the spillway. Rock is quarried and crushed up to 2½-in. maximum size. Sand is obtained by passing crushed rock through a cone crusher.

"Concrete is delivered along the face by 2-cu. yd., side-discharge cars hauled by a gasoline locomotive over a track constructed along the horizontal recesses of the placed rock. From the cars it is spouted by means of chutes directly into the cut-off wall and the 60-ft. panels.

"That portion of the facing below the top of the intake tower is to be permanently covered with two layers of 1½-in. planking. This timber covering was used in place of a form and the concrete facing poured directly behind and against it. For the panels above this level, sliding steel forms, 8 ft. wide and 62 ft. long, will be utilized. These consist of a plate stiffened and supported by two latticed trusses. Each 60-ft. panel will be completed in one continuous operation, the forms being moved while pouring is in progress."

Economic Factors Affecting Choice of Rock-Fill Dam

"The possibility of an economic power development depended upon the cost of storage at the Salt Springs site. The rock-fill type of dam finally chosen appeared to be the structure best suited for this location.

* * * * * * *

"The chief factors favoring a rock-fill dam at the Salt Springs site were the abundance of excellent granite conveniently located for quarrying, the relatively low total tonnage to be transported (less than 20 000 tons), and the fact that construction could proceed, although at reduced speed during the winter months, throughout the entire year.

* * * * * * *

"A concrete gravity dam at this site would have contained more than 800 000 cu. yd. of concrete, requiring the hauling of 152 000 tons of cement, or twenty times the tonnage of cement to be hauled for the present structure. It is estimated that a concrete structure at this site would have cost approximately $1 500 000 more than the rock-fill dam."

DISCUSSION

COMMENTS ON DESIGNS

By D. C. Henny,* M. Am. Soc. C. E.

Statistics showing the trends in the construction of rock-fill dams with tight diaphragms were given by Mr. Henny, in the following tabulation:

Item. No.	Dam.	Height, in feet.	Water slope.	Dry slope.	Top width, in feet.	Tight element.	Reference.
1	Walnut Grove, Ore..	110	0.5 to 1	0.6 to 1	15	Double, 3 by 8-in. timber facing	*Engineering News*, 1888.
2	Lower Otay, Calif...	130	1 to 1	1 to 1	20	Center steel core	*Engineering News*, 1898.
3	Relief Dam, Calif....	140	0.5 to 1	1.5 to 1	13	Concrete facing, 3 to 9 in.	*Transactions*, Am. Soc. C. E., Vol. LXXV.
4	Moreno Dam, Calif..	150	0.9 to 1; at upper 30 ft., 0.5 to 1	1.5 to 1 (21-ft. berm).	16	Rubble masonry, 7 ft.	*Transactions*, Am. Soc. C. E., Vol. LXXV.
5	Dix River, Ky.......	275	{1.2 to 1 {1 to 1	1.4 to 1} 1.2 to 1}	20	Concrete facing, 18 to 8 in.	*Engineering News-Record*, 1925.
6	Salt Springs, Calif...	328	1.3 to 1	1.4 to 1	15	Concrete facing, 36 to 12 in.	....

* Cons. Engr., Portland, Ore.

As to dimensions, Mr. Henny called attention to the fact that Item No. 6 is the most conservative because of its comparatively flat slopes.

Friction Coefficient and Sliding Factor.—The friction coefficient of loose rock on solid rock, or of loose rock on loose rock, is not definitely known and according to Mr. Henny,

"Small-scale experiments which can have little value indicate that it may range from 0.6 to 1.0. If angles of repose in dumping are any indication, a $1\frac{1}{2}$ to 1 slope would correspond to a friction coefficient of 0.67 and a $1\frac{1}{4}$ to 1 slope to one of 0.80; but conditions of sliding under heavy loads are quite different from those affecting the angle of repose.

"The sliding factor in the Lower Otay Dam was far higher, and must have approached the coefficient of friction much closer than that in any of the other dams listed. This was due to the fact that the tight diaphragm did not rest on the water slope which would have caused the entire weight of the dam to oppose sliding. In this case the tight diaphragm consisted of a central vertical steel core against which it may be supposed that full horizontal water pressure was exerted through the up-stream rock. This core was supported only by the down-stream half of the dam, making the conditions opposing sliding comparatively unfavorable.

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"Dams like the Salt Springs have a large margin of safety against mass sliding. It is important, however, to bear in mind that, while the average sliding factor along the base may be low, the local sliding factor varies materially.

"Near the up-stream toe the factor depends solely on the inclination of the up-stream slope. It reduces rapidly in a down-stream direction as may be noted from the following table of approximate values for the maximum 328-ft. section of the dam based upon the assumption that the weight per cubic foot of rock-fill is 96 lb.:

Distance from up-stream toe, in feet.	Local sliding factor.
0	0.77
10	0.72
50	0.58
100	0.45
200	0.29
400	0.11
600	0.03
901	0

"The table indicates that should it happen that the friction coefficient is 0.60 this value is exceeded by the local sliding factor for the up-stream 40 ft. of the base. This would mean that when the water load comes on, the rock fill near the toe would, by horizontal movement, take its set against the material down stream for which the sliding factor drops below the coefficient of friction.

"Such horizontal settlement would be apt to cause cracking of the concrete facing near the toe unless prevented by flexibility or by a horizontal sliding joint."

That this statement should imply that a crack at the toe is certain to occur, was disclaimed by Mr. Henny, because there is no way of knowing certainly that the friction coefficient will exceed 0.77. Rather, it was his intention to point out that the factor of safety against such occurrence cannot be far from unity. If it should be less than 1, said Mr. Henny,

"The result might easily be annoying as leakage through a crack would acquire high spouting velocity, which in time might cause erosion, displacement of back-fill, and further cracking.

"To the writer it appears that serious attention should be given to means which may prevent toe cracking due to horizontal settlement along the base. To that end the simple expedient may be considered of introducing a curved section of pavement at the up-stream toe, concave to the water side. This may be designed to secure any reasonable factor of safety against local sliding. Assume, for instance, that a value of 0.60 for maximum sliding factor is deemed to afford satisfactory safety, then a curve could be introduced in the slope lining, tangent at its upper end to the present slope at a point about 40 ft. from the toe, measured along the slope, and tangent at its lower end, where it strikes the base to a line with a slope of 1.67 to 1. This change involves but a relatively small amount of extra concrete and rock fill and would, the basic assumption being correct, make horizontal movement impossible."

IMPORTANT CONSIDERATIONS IN THE DESIGN OF ROCK-FILL DAMS

By L. F. Harza,* M. Am. Soc. C. E.

A discussion of Mr. Peterson's paper was read for Mr. Harza by Frederick H. Fowler, M. Am. Soc. C. E. Mr. Harza introduced his remarks with the statement that:

"The success of a rock-filled dam is dependent upon:

"1.—Ample spillway.

"2.—Conservative slopes.

"3.—A water-tight diaphragm of such flexibility as to insure against cracking under settlement conditions."

He seemed satisfied that these three requirements had been amply considered in the design of the Salt Springs Dam. The Dix River Dam, in Kentucky, offered many convenient points of comparison and Mr. Harza took advantage of these in discussing the subject presented by Mr. Peterson.

For example, he pointed out that the maximum floods of the two drainage areas were about the same. At Dix River, trouble was encountered in constructing a facing slab without an irregular surface due to settlement. Mr. Harza inquired as to how this was guarded against at Salt Springs. Furthermore, he said,

"It would be interesting to know from the author as to what swell has been measured in the rock-fill at Salt Springs as compared with the measurement in borrow-pits. At Dix River, this was about 40 per cent.

"Settlement at Dix River Dam has practically ceased in a period of four years with a maximum settlement near the center of the dam of about 1.7 ft. in a fill having a height at this point of 275 ft. above stream bed. This is equivalent to a settlement of 0.62 per cent."

Commenting on another important phase of the experience gained at Dix River Dam, Mr. Harza declared,

"The grooves in the dry masonry walls proved an expensive and difficult element of the construction at Dix River Dam. Moreover, there is a funda-

* Cons. Engr.; Pres., Harza Eng. Co., Chicago, Ill.

mental error in principle in back-filling these grooves with concrete poured solid against the walls of the grooves.

"If the vertical expansion joint on the concrete rib is to function by closing up as the result of settlement or expansion, then there must be freedom of movement of the facing slab with reference to this concrete rib, and with reference to the supporting rock adjacent to this rib on either side. The concrete rib, poured directly into the rock grooves and against the rock forming the side walls of this groove, prohibits any movement of the adjoining rock. Therefore, if any value is to be had from the 3-in. contraction space provided along the center line of the ribs, the slabs must break free from the adjoining rock and slide over them. To function properly, the rock next to the rib should be able to move $1\frac{1}{2}$ in. without coming in contact with the concrete rib. * * * In reality, with the concrete rib poured solid into the groove, the rock next to the rib is prevented from experiencing any movement whatever; and the freedom to move increases toward the center of a panel, whereas the tendency to move decreases. The result is that the rock which should move the most in order to permit functioning of the expansion joint is unable to move at all.

"This fallacy of principle was discovered during the construction of the Dix Dam, and for the latter part of the construction a rather unsatisfactory attempt was made to provide a space between the rib and the rock adjoining it on either side. Even this is not ideal, since a large rectangular rock thoroughly packed in with spalls is not very free to move because of interlocking with larger rock and breaking joints with larger rock below it and on either side. The correct principle theoretically would be to have a space of, say, 2 in. between the concrete rib and the adjoining rock on either side, and also to have a division down through the dry laid masonry wall along the center line of the rib extending far down into this masonry wall or entirely through it, this expansion space to be approximately 3 in., or the same as the expansion space allowed between concrete slabs. This would permit the joint on the rib to function freely without breaking the slab loose from the face rock. Another method would be to omit packing with spalls in the vertical spaces between the face rock, packing only the horizontal joints to support the weight. On the Strawberry Dam, a layer of mortar was first spread over the surface of the rock and screeded smooth, the face slab then being poured upon the same, thus furnishing freedom of movement except as restrained by friction.

"These considerations must make it evident that the expansion or contraction joints along the ribs in rock-fill dams, as now built (except Strawberry Dam), must inevitably function, if they do at all, only by the concrete slab breaking loose from the laid wall underneath and riding over these rocks in their movement, thus, after a movement, failing to conform to the surface of the rock wall underneath. Since they have nearly all been built this way, it is no reflection on any one to suggest this incorrectness in principle, which apparently still awaits a simple solution.

"It is our belief that the infrequency with which these joints have been observed to function indicates that the adherence of the slab to the rough rock surface is sufficient to compel the slab to act in general as a monolith even across the joints, thus largely defeating the primary purpose of these joints. In cases where the thrust becomes sufficiently great, the slab can break loose from the rock and the joint can function. It is, therefore, of some value in extreme cases, although only by injury to the uniform support of the slab."

In conclusion, Mr. Harza stated,

"With ample spillway and conservative slopes, the failure of a rock-filled dam in the sense that other dams have been known to fail is inconceivable. It is conceivable that cracking of the face slab, due to poor construction, might

induce sufficient leakage to be a matter of concern economically; but with a properly constructed fill, it is not conceivable that this leakage could reach such proportion as to cause failure of the dam, or hazard to the valley below. There is probably no safer type of dam."

COMMENTS ON THE DESIGN OF SLOPES

By E. W. Kramer,* M. Am. Soc. C. E.

As a representative of the Federal Power Commission, Mr. Kramer had the opportunity to study the plans and methods of constructing the Salt Springs Dam. In his discussion he stressed particularly the problem of determining the road slopes. Quoting,

"The steepest slope considered was 1.3 horizontal to 1 vertical, and the flattest, 1.5 horizontal to 1 vertical. Measurements were made by the speaker at the site where some quarrying had begun, to determine the natural slope of this rock. This slope was about 1.3 to 1. The argument in favor of a flatter slope than this was that if the exact natural slope were adopted for the down-stream face, and if settlement occurred in the down-stream direction, due to water pressure in a greater ratio than vertical settlement due to weight of the rock, the down-stream slope would become greater than the natural slope and sliding might occur.

"On the other hand, it was realized that it would be extremely difficult to place the rock actually on an angle flatter than its natural slope. The cheapest method, of course, in placing the rock on the down-stream face is to dump it, allowing it to roll down and taking its natural slope. To drag this rock out to a flatter slope would be expensive. The solution arrived at was to adopt an over-all slope which should not be steeper than 1.4 to 1, this general slope being obtained by berms, if necessary. The rock was to be dumped into place without effort being made to flatten the natural slope. A slope of 1.3 to 1 was adopted for the up-stream face for several reasons:

"First, that this is approximately the natural slope of the rock, thus allowing the rock to be dumped in place and to settle to a considerable extent before the derrick-placed rock is laid.

"Second, a slope steeper than the natural slope would require that the rock placed by derrick be carried up with the dam.

"Third, the added expense of a slope flatter than the natural slope could not be justified since the natural slope would be entirely stable, with a very high safety factor, especially after the derrick-placed rock has been laid."

A slope steeper than the natural was never considered in this connection, but Mr. Kramer propounded an interesting question, namely,

"Is it more economical and more desirable to use a steep slope which requires a thicker facing of derrick-placed rock, and also placing the face before the initial settlement has occurred, than to use the natural slope of loose rock with a thinner facing of derrick-placed rock placed after the initial settlement has taken place?"

Answering the question, he said,

"In so far as the stability is concerned it is believed the argument is all in favor of the latter method. However, there may be sufficient economy in the former method to justify its adoption in dams of lesser height, and perhaps even in dams of this height where the rock is not so easily available.

* Dist. Engr., U. S. Forest Service and Federal Power Comm., San Francisco, Calif.

"Another question of interest that came up during the progress of construction was as to whether or not it would be safe to allow the natural river fill, consisting of large boulders, gravel, and sand, to remain in place and to deposit the rock fill on it. It was intended, of course, in any case, to carry the cut-off wall and concrete facing wall into the bed-rock. There were about 200 000 cu. yd. of material in the stream bed. This was a river fill consisting of rocks varying in size from 5 yd. down to river sand. It was free from clay loam or humus. A very large percentage consisted of large boulders. A cross-section of this material showed pockets containing up to 1 cu. ft. of sand, which prevented the larger boulders from making actual contact.

"The material was about 20 ft. deep just below the up-stream face of the dam and had a slight surface slope toward the center of the stream and also down stream, although it maintained about the same vertical thickness. It extended entirely across the stream bed and up and down stream beyond the dam foundations. It has been explored with numerous drill holes and several test pits.

"Estimates of the weight of the dam on this material were from 15 to 30 tons per sq. ft., according to the bearing surface of the rocks placed on it. The material was thoroughly compacted, and would take this pressure with an immaterial settlement, provided it was confined.

"Two problems presented themselves: What would be the effect of any considerable leakage through the face of the dam running over this material? What would be the effect of a small leakage through the face, piling up between the cut-off wall and the material, and seeping through the material under a head of possibly 20 ft.?

"The material differed from the fill of a rock-fill dam in that there was a larger percentage of sand. It was in a different condition from the fill of a rock-fill dam in that there were sand pockets between the rock surfaces. In an ordinary rock fill, as deposited and washed with a water jet, the contact of rock on rock is assured since the percentage of sand is small and the sand is washed away to a point where the rock already has good contact. The large rocks of an ordinary rock fill will allow for a large leakage, which will run through freely without damage, whereas the material in question, while slightly pervious as a tight sand fill might be, might allow a pressure to be built up inside of it which would cause a slight but continuous sluffing on the down-stream face.

"A still worse possibility was that channels might form through it which, although of moderate size, might wash out the sand pockets, and cause enough settlement to open the concrete face sufficiently to cause a failure."

One construction problem of some magnitude was that encountered in attempting to control unusual floods that might occur. The method proposed, according to Mr. Kramer,

"Is to divert the stream flow by means of a coffer-dam, or the finished face of the rock-fill dam as fast as it is laid, through a tunnel 22 ft. in diameter and about 1 200 ft. long, which tunnel will be used, after construction is completed, as the intake for the conduit leading from the reservoir to the power house.

"Adopting a method of this kind places reliance on the construction program, the amount that the tunnel can carry being dependent on the head placed upon it."

GENERAL DISCUSSION

When two such structures as the Salt Springs Dam (328 ft. high) and the Dix River Dam (275 ft. high) are being compared, certain characteristics of

site and material must be taken into account. This thought was expressed by C. E. Grunsky, Past-President, Am. Soc. C. E., who stated that,

"In the case of the Dix River Dam there is a canyon washed out in limestone formation. The limestone has broken away with vertical faces stepped off from the surface down slab after slab to the bed of the creek. The material of which the Dix River Dam is built is limestone.

"In the case of the Salt Springs Dam, the side slopes are gradual. The material of which the dam is being constructed is good, firm, hard granite.

"Will the settlement at the Salt Springs Dam be the same as the settlement of the limestone material at the Dix River Dam? It cannot be told in advance."

There will be settlement, said Mr. Grunsky, generally about in proportion to the vertical depth. Therefore, he said,

"It is always desirable to make the up-stream face of the rock-filled dam as steep as possible in order that it may not break away from its support.

"The site with the most gradual uniformly sloping sides is the ideal site for the rock-filled dam. The less favorable site is the site of the type at Dix River where there are vertical faces against which the material is to rest. The facing slab is much more likely to break away from these vertical sides than it is in the case of a gradual sloping surface."

H. K. Fox, M. Am. Soc. C. E., outlined his experience in examining two existing dams. These observations led Mr. Fox to believe that the natural slope of rock is sufficient for a rock-fill dam.

One of the dams was a small structure of only about 50 or 60-ft. head, constructed about 1890. Mr. Fox believed that it might have been constructed by a logging foreman, with the help of a cableway. The rock was placed apparently as steep as it could stand. Upon examination Mr. Fox found that below the water-line the timber facing was as good as new and that there was no evidence of sliding.

The other dam cited by Mr. Fox was 140 ft. high, constructed about 1913. This dam has a concrete core wall in the center and the up-stream part is mud-sluiced. The down-stream section is rock-filled (granite). In commenting on this case, Mr. Fox declared that the dam,

"Has been filled within 4 ft. of the top, probably one out of every three or four years since.

"In its early days it was noticed that it was moving. The engineers established points on the top of this rock-fill and then also established two permanent points on the ends and every year a record was kept of the movement by observation of the transit on those points. We checked those records carefully many, many times and there has been a great deal of discussion as to in what manner the dam at the top of this core wall was moving."

A shaft was dug and this disclosed that,

"There was no horizontal crack in this core wall until you reached the bottom and there is apparently one horizontal crack at that point. In fact, we proved that there was and we drilled into it and found it. The slope of the core wall was uniform. In other words, it was not as I personally had expected, that there would be several cracks and it would perhaps form a curve instead of a straight line. It was as uniform as one could expect it to be.

"We examined the foundation. We went down until we uncovered the bed-rock, and knew that we had the original stream bed, because you could see

the marks of the erosion of the stream. The granite was polished smooth and when we washed it off it was the same as when the dam was built. We could see no signs whatever that the rock had moved horizontally, and there was no sign of any shearing of the core wall itself.

"One of the things that was expected, that perhaps this core wall had sheared right at the rock surface, the original stream bed, was not the case. It had cracked horizontally about 7 or 8 ft. above the stream bed, and I attribute that to the fact that the lower section was a little thicker. In other words, they stepped the core wall in at that point. It was 7 to 8 ft. thick at the maximum point."

Finally, said Mr. Fox,

"We made many cross-section surveys of both the up-stream and down-stream slope and concluded to add rock. We did recently add more than 100 000 yd. We have not had the dam full since that time, nor the reservoir full, so we do not know whether or not that has stopped this gradual movement. But it did prove to us that the movement was not due to sliding on its foundations, and that it was due to a gradual settling and disintegration of the material. This granite is not as good a grade as they have at Salt Springs. I believe that, if it was, the settlement would have been much less."

A rock-filled dam, 125 ft. high, recently built in Mexico, was described by Louis C. Hill, M. Am. Soc. C. E. The slopes were given as 0.7 to 1 and were made with hand-placed masonry. Quoting Mr. Hill,

"It was originally designed to be hand-placed masonry on the up-stream side, that is, dry masonry on the up-stream side, and the down-stream side connected with a series of walls intersecting, and the inside of these walls filled with loose rock; but I understand it was actually constructed by being laid up as a dry rock-fill. That is the cheapest front and back slope that I have ever seen."

Mr. F. C. Finkle recalled having considered building a rock-filled dam as high as 450 ft. and asked Mr. Peterson if the preliminary studies of Salt Springs Dam had considered the question of maximum limitation of height. In the case mentioned by Mr. Finkle the 450-ft. height was found to be economically impractical.

From the discussion of A. H. Markwart, M. Am. Soc. C. E., the impression was gained that no such studies had been made since (quoting Mr. Markwart):

"There is one thing that the engineers of the Pacific Gas and Electric Company do not attempt, and that is to be the first to do anything unnecessarily. We would rather play second. We found that the dam would have to be upward of 300 ft. high from the stream bed and, perhaps, 30 ft. more to suitable bottom and it worked out some 328 ft.

* * * * * *

"Answering the question about the limit of height: We based it largely on common sense and precedent. I think one could go higher, but the greater settlement problem would come into play. I think that you can go almost any height with good rock if you wait long enough to put 'the skin' on it. That is difficult where you are building in seasons because you cannot wait very long as it is difficult to get rid of flood waters that come during seasons. If, however, you make adequate provision for taking care of all the floods that might come over that period that structure is settling, I think you can do anything you want, because rock, in a natural state, will carry itself very high.

"I suppose we can make calculations on what granite will carry, but the difficulty is to figure out how much will be on any one point of rock. So a

scientific analysis of what granite might carry would not be of much value. The greatest value is precedent, what other dams have done."

In the main, Mr. Markwart's remarks were in the form of a closing discussion of the paper by Mr. Peterson, in so far as the more important questions raised were answered. For example, in reply to Mr. Henny on the subject of possible "horizontal settlement," Mr. Markwart explained,

"There are a number of factors that would safeguard us in case of that horizontal compacting. First of all, about one-half that 50 ft. is laid-up rock, not rock-filled. Again, there is considerable concreting in the upper portion of the dam. Furthermore, about 80 ft. vertical of the concrete base is covered with boards, which are left in place, which have considerable flexibility, and sooner or later we expect to sluice in about 30 ft. of earth material in front of the dam at the lower level. Then, of course, last but not least, are the flexible horizontal joints, or the flexible horizontal joints across the dam, which can have some movement that would be brought on by this horizontal compacting."

Calling attention to Mr. Harza's discussion relating to grooves in the dry wall and especially to his recommendations, Mr. Markwart stated that:

"In the writer's opinion, if the face of a dam is to be constructed in panels, with the reinforcing crossing the joints to form a continuous slab, as at Bucks Creek, or with the panels separated as at Salt Springs, some form of rib should be used to form a rigid support for the ends of the slab at these construction joints, although there perhaps would be less necessity for this with the continuous slab construction. Otherwise, if local settlement occurred near the ends of the slab, the slab would be subjected to cantilever action, which would produce stresses five or more times as great than for a similar area of local settlement some distance from the edge of the slab. The concrete rib, rigidly tied in to the adjoining rock around the boundary of the panel, produces a support which is less yielding than the rock in the area it encloses, thereby greatly reducing the possibility of local settlement in the placed rock in the areas where such settlement would damage the slab the most. I believe that such supports should be provided behind any construction joint on a dam of this type, where settlement is involved.

"When the rib is poured into and around the adjoining rock, as at Salt Springs, and a space of 1 in. is left on the inclined joints between each rectangular panel and when the slab itself is poured directly on the rough area of the derrick-placed rock, it must be admitted that this joint can only be closed by some shearing of rock points or of the projections of concrete which find their way back into the voids between the placed rock. If the space between the concrete panels is large, as described by Mr. Harza, this shearing action would extend some distance away from the rib, but where the space is only 1 in., making it necessary to provide for a movement of $\frac{1}{2}$ in. on either side of the rib, it is quite likely that all this small movement can take place in the contact between the placed rock one course removed from the keyway. In this case, only one or two of the concrete or rock projections would be sheared or crushed in order to permit movement and to allow the slabs to contact with each other.

"If this shearing, which occurs only if the slabs move with respect to each other, is considered objectionable, a very simple preventative would be to make the face of the rock smooth for perhaps 5 or 6 ft. on each side of the joint, to prevent adhesion to the concrete.

"The construction of a detached face, as used for Strawberry Dam, was considered for Salt Springs Dam, but was eliminated because of the great area

of the face. An element of this slab is about 530 ft. long measured from cut-off to crest and its resistance to column action would be somewhat problematical.

"The preservation of the integrity of the water-tight face as the dam takes settlement is, as Mr. Harza says, one of the three important features necessary to insure the success of a rock-fill structure. The relatively small settlement of the monumental Dix River Dam and the excellent manner which the face of that dam has conformed to this settlement, inspires a feeling of confidence in high rock-fill dams where there may previously have been some apprehension. In our Salt Springs Dam, we are stepping up the height of rock-fill dams another 50 ft. and hope through the excellent quality of the native rock and high character of construction, successfully to demonstrate that even greater heights are possible.

"I concur in Mr. Harza's statement that with ample spillway to insure against overtopping, with conservative slopes to prevent sliding, and with a facing which can settle with the dam without cracking injuriously, so that blow-outs cannot occur, there is probably no safer type of dam."

SURVEYING AND MAPPING DIVISION

April 24, 1930—3:10 P. M. to 5:45 P. M.

PRELIMINARY TOPOGRAPHIC SURVEYS FOR PROPOSED COLORADO RIVER AQUEDUCT

By E. A. Bayley,* M. Am. Soc. C. E.

This was declared by the author to be one of the most comprehensive topographic surveys ever undertaken in this country by an agency other than the Federal Government. The area covered was more than 24 000 sq. miles and the cost to date has been $1 750 000.

Much valuable experience was gained from similar surveys for the Owen's River Aqueduct, and yet, according to Mr. Bayley, some practices followed at that time were found not adaptable to the later conditions.

Quoting Mr. Bayley,

"On the preliminary survey for the Colorado River Aqueduct, a very comprehensive system of triangulation was adopted for the control of all preliminary and location surveys. This triangulation extended over the entire area, from the Mexican boundary line to Utah on the east and to Los Angeles on the west. For its base lines, the system was tied to the Texas-California Arc of Primary Triangulation of the United States Coast and Geodetic Survey. It was also tied to the triangulation system of the United States Geological Survey wherever available. Thus, a single standard of control for all subsequent topographic and location surveys was made available for the field engineers' use, and it is hoped that by continually tying to this triangulation system, none of the work now completed or in progress will, in future years, be lost to engineers having need for it."

Preliminary Reconnoissance

After a preliminary reconnoissance it became apparent that the San Gorgonia Pass would be the most logical place to begin the survey. Accordingly,

"Palo Verde Valley, through which the Colorado River runs, was the nearest point to San Gorgonio Pass; therefore, the mapping of the intervening territory between these two points was considered the first thing that should be done. The work was started in this area, and as the survey advanced, it became apparent that an advantage would be gained by mapping the entire territory lying within the approximately triangular area between Boulder Canyon on the north, Yuma on the south, and Los Angeles on the west, as pumping would be required along any route the intake of which was located on the lower portion of the river."

* Engr. of Surveys, Dept. of Water and Power, Los Angeles, Calif.

The preliminary field surveys included a triangulation network for horizontal control, lines of levels for vertical control, topographic mapping, etc. A field supervising engineer directed this work, at times with the help of one or two assistants. According to Mr. Bayley, the various parties established their quarters in the nearest town or settlement wherever possible. In some cases camps were necessary, and these were usually established at the nearest well or spring. "Dry" camps were sometimes established and then water-hauling was necessary.

Field headquarters were established at Beaumont, Calif., on the summit of the San Gorgonia Pass. All the details in connection with the field work were handled at this place, under the direction of an assistant engineer, thus relieving the Los Angeles Office of much work. Mr. Bayley described the organization of this office as follows:

"The engineering parties received their instructions from this office and on completing their assignments, turned in the original records of their instrumental work to the field headquarters.

"A complete system of indexing and filing was maintained at Beaumont during the entire period of the survey, with the result that all the original data pertaining to horizontal and vertical control were easily located when desired. In addition, a blue print for each triangulation station, showing both the spherical and rectangular co-ordinates of the station, together with the azimuths and lengths of lines radiating therefrom, was made available for the use of the field engineers.

DEPARTMENT OF WATER AND POWER
CITY OF LOS ANGELES

D— 86
S— 9

DESCRIPTION OF TRIANGULATION STATION

NAME OF STATION GANNON DATE April 1929

COUNTY San Bernardino STATE California

APPROXIMATE LOCATION Fenner, California, 20 miles West of.

DESCRIPTION Station is located on a prominent peak on the southeast end of the Providence Mountains. May be reached by driving 20 miles west from Fenner on the Fenner-Kelso road and then turning northwest on the Gannon Camp Road. Following this road for 3 miles, a deep, narrow canyon will be seen on the northeast. Drive up this canyon 500 feet. From this point, walk up the canyon 1000 feet and then climb the east wall, a comparatively easy 30-minute climb. Approximate Elevation 4600 feet.

SPECIMEN

STATION MARK 1" Iron Pipe set in concrete with 2"x2"x10' pole in 5' cairn.

ESTABLISHED BY J. D. Taylor DESCRIBED BY J. D. Taylor
CHIEF OF PARTY

ILLUSTRATION OF CARD BEARING DESCRIPTION OF TRIANGULATION STATIONS AS KEPT IN THE BEAUMONT, CALIF., OFFICE FILES.

"All triangulation computations and figure adjustments, and all adjustments of level circuits were made in the Beaumont Office. In computing the triangulation, every precaution was taken to insure accuracy. The various observed figures were always adjusted, the larger ones by the method of least squares. The latitudes and longitudes of all stations were computed by

geodetic formulas, which are adapted to the spheroid representing the earth's size and shape. All distances determined by triangulation computation were reduced to sea level."

Triangulation System

Comparing the surveys for Owen's River Aqueduct with those for the Colorado River Aqueduct, Mr. Bayley explained that for the Owen's River project both chained traverse and triangulation methods of control were used. Although the project was 200 miles in length, the total vertical range in elevation was not more than 250 ft. as compared with about 3 000 ft. on the Colorado River project. Therefore, for the latter survey,

"In order to include every possible combination of pump lift, an area of several thousand square miles would have to be carefully mapped. For this extensive area the triangulation system of control seemed to be by far the best suited and was adopted. With the exception of one short stretch of about 30 miles of chained survey, no traverse control was found necessary for the preliminary topographic mapping of this project.

"As a foundation for the preliminary triangulation work on the Colorado River Aqueduct project, the Texas-California Arc of Primary Triangulation as established by the United States Coast and Geodetic Survey was used as a geodetic control. The stations at the westerly end of this arc of primary triangulation were found admirable as bases for the Colorado River Aqueduct triangulation system.

"Along the course of the Colorado River between Needles, Calif., and Yuma, Ariz., the United States Geological Survey has established an extensive system of triangulation for the control of its topographic mapping in the river area. This system was tied into as often as possible and was found very valuable in controlling our more detailed surveys along the river.

"The Geological Survey also has an established system of triangulation in the Coastal Plain of Southern California. This system is particularly extensive in Los Angeles County and was often used.

"In comparison with the early surveys on the Owen's River Aqueduct, where many Polaris observations were taken for azimuth control, it is interesting to note that no astronomical work whatever was found necessary on the Colorado River project. With many triangulation stations established in this area by both the Coast Survey and Geological Survey and with published records of positions, as well as azimuths and back azimuths between stations, the determination of position and azimuth in any locality became very simple.

"All surveys on the Colorado River project have been based on the final North American 1927 datum of the Coast and Geodetic Survey, which was made available for our use in 1927. This is the final adjusted datum for all triangulation in the western half of the United States. Triangulation adjustments which were made in our Beaumont Office prior to 1927, were recomputed and based on the 1927 datum.

"The standard datum for elevation has been taken as mean sea level and the elevations shown on all plane-table sheets are based on that datum."

Vertical Control

"The bench-marks as established by the Coast and Geodetic Survey and the Geological Survey have been made use of for level control. The elevations used for these bench-marks were those shown in the published *Bulletins* for such bench-marks.

"As a standard of reference, the particular Coast and Geodetic Survey bench-mark located in the small park at the Harvey House, San Bernardino,

Calif., has been taken as the principal bench-mark for the Colorado River project and its elevation as published by the Coast Survey is the basis for all other survey levels."

Instruments and Instrumental Methods

Ten-second and twenty-second transit theodolites were used on all triangulation work. Angles were read by turning repeated angles directly over the center of the station. According to Mr. Bayley, the number of sets of observations and the number of repetitions in each set varied with the requirements of the survey. In his own words,

"The station mark itself almost invariably consisted of a 1-in. iron pipe with cap set in concrete. A small hole drilled in the center of the cap marked the center of the station. Either a 2 by 2-in. pine mast, varying in length from 8 to 14 ft., or a rock cairn, depending upon the length of lines to be observed, was erected over the station mark. When the pine mast was used, it was plumbed over the center of the station and guyed by four wires. On the top of the mast a bonnet of canvas or muslin flagging was constructed for use as a target.

"Sensitive 18-in. or 20-in. engineers' wye-levels were used in most cases for level control, although dumpy levels were used in some instances. All level lines of importance were double-checked and the errors in elevation distributed between initial and terminal Government bench-marks. Several of the level circuits were more than 100 miles in length. In every instance closures on Government bench-marks were closer than required for the topographic mapping which followed.

"The accuracy of the main triangulation system on the Colorado River Aqueduct project is represented by a closing error in a triangle which did not exceed 6″ of arc, and had an average closing error of approximately 3″. This degree of accuracy is approximately that obtained by the Coast and Geodetic Survey in its second-order triangulation work, and was fully sufficient for the primary purpose of control in preliminary topographic mapping."

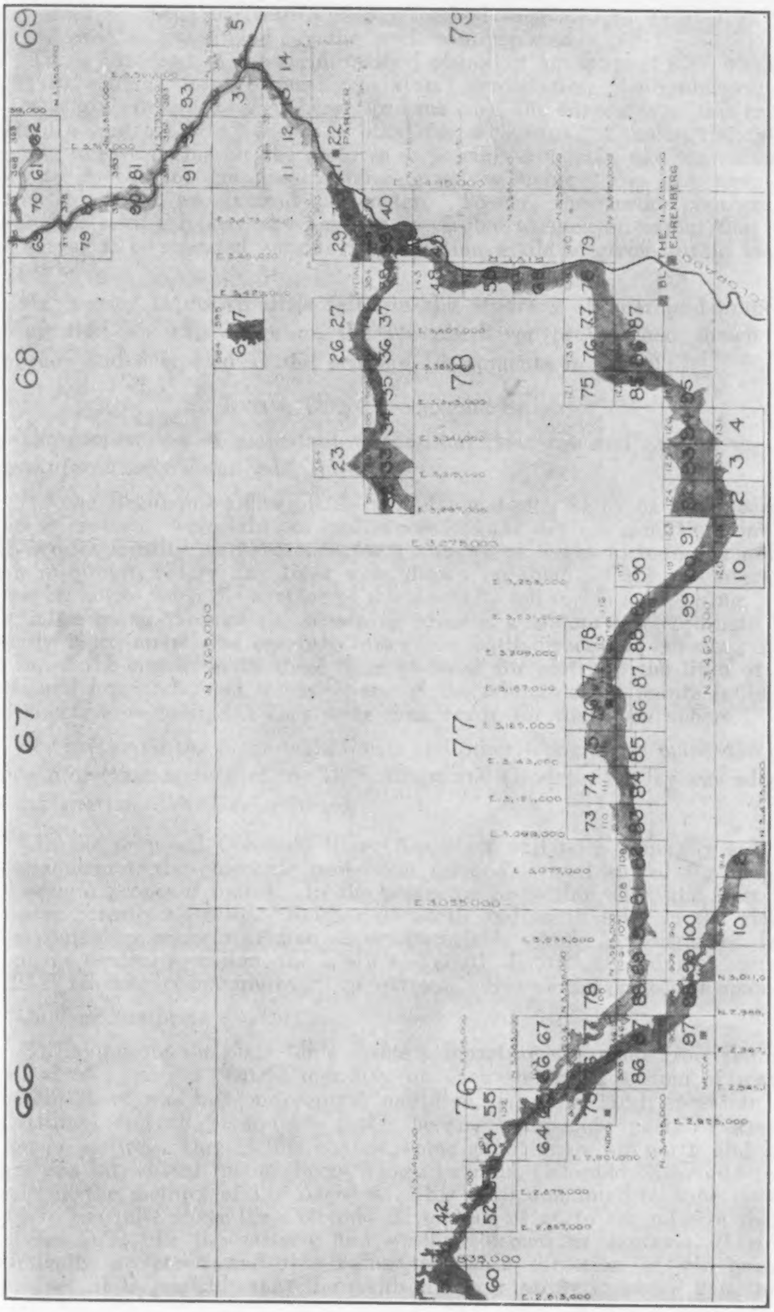
The topographic details were recorded by the plane-table method. Mr. Bayley explained the reason for this choice in detail. For example,

"Much of the territory was mapped on scales of 5 000 and 10 000 ft. to the inch, using a 50-ft. and a 100-ft. contour interval, respectively. For detail work covering narrow belts of topography along projected routes, the much larger scale of 1 000 ft. to the inch was used. Maps on this scale usually showed contour intervals of 20 ft., although in some instances, where the ground surface was quite flat, 10-ft. or 5-ft. intervals were used.

"Before the plane-table method was adopted for taking topography, a great deal of thought was given to the use of other methods. Transit and stadia methods would involve the taking of voluminous notes and the burning of much midnight oil in interpreting and plotting such notes. Consequently, this method of topographic mapping was eliminated.

"Some thought was given to the practicability of the photographic method for taking contours. The adoption of this method would involve the purchase and use of special cameras and the taking of an immense number of photographs. These would have to be properly oriented for position and elevation. There would also be involved the solving of a great quantity of problems in descriptive geometry in order to interpret properly the photographic results. This would make the method highly impractical for the large amount of work to be undertaken.

"Considerable thought was early given to the use of the airplane in phototopographic mapping from the Colorado River to Los Angeles. At the com-



PORTION OF INDEX MAP TO ILLUSTRATE ORIENTATION OF 1 000-FT. SCALE, TOPOGRAPHIC SHEETS ON 10 000-FT. SCALE SHEETS, OUTLINING AREA TAKEN ON 1 000-FT. SCALE.

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mencement of these surveys in 1923, the aerial photo method of taking topography had not been perfected. Every method suggested to us at that time was rejected as unsuitable for the work contemplated.

"Those advocates of the aerial method of taking topography, with whom we came in contact, were primarily aviators; secondarily, photographers; and lastly, engineers and topographers. In each case, the advocates of this process desired a contract of some sort or other for such work. Usually, they would propose to give us merely the pictures or possibly a mosaic, and thereafter we were to do the topographic mapping ourselves, using these photographs, a plane-table, and an aneroid barometer. To-day, the aerial photographic method of taking topography has been perfected to such an extent that were the survey to be repeated, serious consideration would be given to that method of work."

Mr. Bayley expressed little faith in the accuracy of aneroid barometers, saying that his experience on the Owen's River project had shown that accuracy had decreased as the price of instruments increased.

Laying Out Topographic Sheets

The preparation of plane-table sheets and their use and care in the field were interestingly described:

"In the Beaumont Office blank plane-table sheets, 24 by 31 in. in size, as soon as received, were laid out and exposed to the dry air conditions prevailing in that locality, and seasoned for a number of weeks to reduce distortion to a minimum before any lines were drawn on them. Care was exercised, however, not to warp the surface of the sheets by too severe a seasoning.

"After going through the seasoning process, a system of co-ordinate lines exactly 2 in. apart was carefully drawn in both directions, covering 22 by 30 in. of the sheet. With these lines as bases for plotting, the lines of latitude and longitude, and the positions of the triangulation points falling on the sheets were plotted. They were then ready for the topographers."

For preparing the plane-table sheets and other topographic maps, the polyconic projection method of the U. S. Coast and Geodetic Survey was adopted. In explanation, Mr. Bayley stated,

"As the proposed Colorado River Aqueduct will be a generally east and west aqueduct, the polyconic projection method is well suited for mapping the several proposed routes. In the polyconic projection, east and west lines show practically no error. Neither do north and south lines in the vicinity of a central or prime meridian show appreciable error. As distances depart from the central meridian, the north and south distortion gradually increases until it reaches its maximum at the extreme edges of the projection mapped."

Quoting further,

"In laying out the plane-table sheets a departure was made from the usual method of having a central meridian on each sheet. A system of platting wherein there was but one central meridian, *viz.*, the 116th meridian, was substituted and all plane-table sheets became component parts of a single large projection. Due to this change, some slight error for north and south lines was introduced in the sheets lying along the Colorado River and in the sheets in the vicinity of Los Angeles. This error amounted to approximately 1 ft. to the mile along the Colorado River and 2½ ft. to the mile in the Los Angeles area. In the easterly and westerly directions, however, there was practically no error, and this being the main direction of the proposed aqueduct, it is probable that the resultant error rarely exceeded 2 ft. to the mile.

"The advantages of having one central meridian for the entire system of sheets were many. First, a system of co-ordinates based on the Coast Survey grid system for progressive maps provided a single comprehensive system of rectangular co-ordinates. This rectangular system was convenient for laying out projections, and for many other purposes. Second, the perfect matching of one sheet with another at the edges could be maintained throughout, and this would be impossible should each sheet have a separate central meridian and a separate polyconic projection drawn on it. Third, when determination of distances between points on adjacent sheets was required, as is often the case in laying out grades, the rectangular system of co-ordinates could readily be used and corrections applied for north and south scale error, if necessary.

"As the edges of the plane-table sheets were laid out by rectangular co-ordinates rather than by spherical co-ordinates, we were enabled to orient the sheets so that, except in the case of special sheets made for the convenience of a topographer when working in the vicinity of a corner, no two sheets overlapped or were staggered. Work done on special sheets was later transferred to the regular non-overlapping sheets.

"By the use of this system of orientation, we were enabled to adopt an ingenious decimal system of numbering plane-table sheets. With the 10 000-ft. scale sheets as a base, numbers increased from left to right, and from north to south in decimal order. As an illustration, the numbering in rows would be of the order 76, 77, 78, etc., and the numbering in columns would be 67, 77, 87, etc. Within each 10 000-ft. scale sheet, the 1 000-ft. scale sheets were similarly shown. Larger scale sheets of 100, 200, or 500 ft. to the inch were shown as subdivisions of the 1 000-ft. scale sheet. A convenience afforded by this system of numbering was that the numbers of adjacent sheets were always known and index references were unnecessary."

Mapping Conveniences for the Engineer

In the ordinary process of writing contour elevations on contour maps, they are made to read from the bottom and from the right-hand side of the sheet. Experience described by Mr. Bayley indicated that a better way was to write all contour elevations uphill. In this way,

"It is only necessary for the engineer busily engaged in handling topographic sheets to observe one elevation, and not even to read that, to determine which direction is uphill, for the way the number faces determines this point. On our Colorado River work, all elevations were written uphill and the convenience afforded was very marked.

"An important item of the preliminary topographic survey for the Colorado River Aqueduct project was the determination of the elevations of all saddles to be shown on the contour maps. These the topographer usually recorded, but occasionally the draftsman erased when cleaning up the sheets after inking, much to the annoyance of the locating engineer, who was about to project a preliminary location to pass through the saddle. On a number of occasions, the loss of a saddle elevation required a return to the field to re-determine such elevation before the projection could be continued.

"A departure from the customary method of accentuating every fifth contour on the topographic maps by making it with a heavier line, was the substitution of a fine, black line, while the intermediate contours were drawn in brown. This gave the same relief and yet furnished a slight additional convenience when drawing grade contours where the contour lines were very close together."

The Los Angeles Office has constructed a large relief map covering the entire territory mapped. Describing this work in particular, Mr. Bayley said,

Illustration of Relief Map Covering the Colorado River Aqueduct Project.

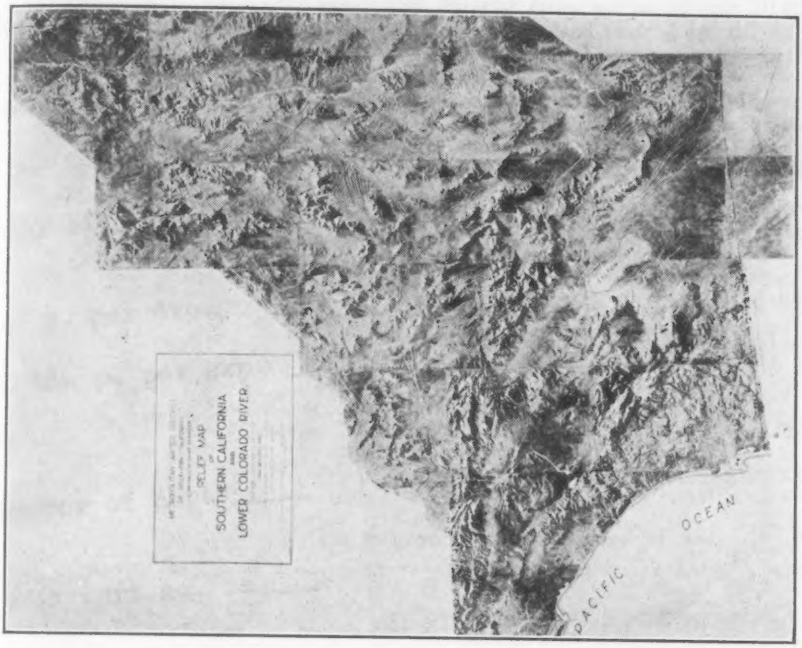


Illustration of a Section of the Relief Map to Enlarged Scale.

"The contours were pantographed directly from prints of the 10 000-ft. scale plane-table sheets on to sheets of red vulcanized fiber, $\frac{1}{8}$ in. thick. The contour interval for the 10 000-ft. scale plane-table sheets being 100 ft., each $\frac{1}{8}$-in. fiber sheet represented 100 ft. in elevation.

"The contour outline on these fiber sheets was then cut to form by means of a jig saw using cylindrical dental blades. After cutting out the contours, the sheets were superimposed one upon the other, and nailed in place with $\frac{3}{4}$-in. brads. In this manner, the various sections of the relief map were built up to completion."

An interesting statistical summary of this work, given by Mr. Bayley, is as follows:

"STATISTICAL DATA—COLORADO RIVER PROJECT, 1923 TO 1930

"(1) *Triangulation.*—

Area, in square miles, furnished with horizontal control (approximate triangle, Los Angeles-Yuma-Virgin River)	27 000
Number of geographic positions determined, exclusive of ties to Government triangulation	2 284

"(2) *Leveling.*—

Total miles run over new ground and exclusive of re-running	3 272

"(3) *Topography.*—

Area topographically mapped:

Scale.	Square Miles.
1 in. = 100 ft.	8
1 in. = 200 ft.	5
1 in. = 500 ft.	15
1 in. = 1 000 ft.	2 732
1 in. = 5 000 ft.	2 252
1 in. = 10 000 ft.	19 644

Number of plane-table sheets utilized for plotting topography in part or in full:

Scale.	Number.
1 in. = 100 ft.	76
1 in. = 200 ft.	28
1 in. = 500 ft.	10
1 in. = 1 000 ft.	370
1 in. = 5 000 ft.	38
1 in. = 10 000 ft.	22

"(4) *Miscellaneous.*—

Longest line, in miles, observed in both directions, Butte-American	59.2
Maximum number of plane-table parties in the field at any one time	23

Main Scheme Control:
Standard second-order triangulation

Topographic Control:
Standard third-order triangulation

Maximum temperature at Indio during the summer of 1929, as officially reported by Government Experimental Station, occurred on June 24	121° Fahr.

DISCUSSION

COMMENTS ON THE TRIANGULATION

By DeWitt L. Reaburn,* M. Am. Soc. C. E.

Usually, said Mr. Reaburn, the routes of aqueducts are fixed by the known position and elevation of the source of supply. This was true of the Owen's River Aqueduct, but the point of diversion from the Colorado River may be anywhere from Yuma, Ariz., to the Grand Canyon. To determine the economically possible routes under these conditions required the preparation of a topographic map of the entire area.

The manner and accuracy with which this was done were discussed by Mr. Reaburn. Quoting,

"The triangulation system for the Colorado River Aqueduct served not only for control of the small-scale topographic maps and the more detailed route maps, but will also be available for control of the final location surveys.

"The adopted route when finally located will, no doubt, contain many miles in tunnel. The measurement of these tunnel lengths over the top by chain is a slow and expensive procedure. By use of the triangulation system these lengths can be readily determined in advance as accurately as they can be measured by chain after the tunnels have been holed through.

"This method was generally used on the Owen's River Aqueduct and no discrepancies in distance were noted when the tunnel headings came together.

"The average triangle closure of 3″ obtained on the Colorado River triangulation would indicate a probable error of about 1 in 10 000, a degree of accuracy probably greater than ordinary chaining under desert and mountain conditions. On the location of the Owen's River Aqueduct, a line of precise levels was run over the entire length of the line, and precise benchmarks were established at frequent intervals to furnish vertical control for the level lines run by the various locating engineers, and for establishing grade elevations on construction.

"The plane-table method and the transit stadia method are both extensively used in topographic surveying. The plane table is indispensable on small-scale work, but on large-scale detail mapping—of 200 ft. to the inch, or larger, where each contour is accurately rodded out—the transit stadia in the hands of an experienced man, will give the best results.

"The art of small-scale topographic mapping requires special training over a period of years, and it was fortunate that men with such training were available for the Colorado River work.

"On location surveys the horizontal control is usually provided by the chained traverse, and the azimuth control by polaris observations taken at intervals along the route of survey.

"Owing to the convergency of meridians, a correction for easting or westing must be applied to the azimuth carried forward through the traverse to determine the true azimuth at any point. The amount of this correction is small when the route of survey is in a north or south direction, and reaches a maximum when the route is in an east and west direction.

"In the latitude of the Colorado River Aqueduct, the convergency amounts to about 35″ per mile of longitude, a total of about 4° between the Colorado River and Los Angeles. Both spherical and rectangular co-ordinate

* Cons. Engr. (Reaburn & Bowen), Los Angeles, Calif.

lines were projected upon the field sheets and the positions of the triangulation stations and azimuths of the lines were computed both on a spherical and rectangular basis, the rectangular azimuths being referred to the 116th meridian."

In plane-table work, the position of the station occupied is determined by a graphical solution of the three-point problem. Mr. Reaburn described an interesting method, devised by him, for an analytical solution similar to the graphical. The analysis follows:

"Let A, B, and C be the three points of known position, and P the point of required position.

"Let the unknown angles at A and C be represented by x and y, and the observed angles be P_1 and P_2. We then have:

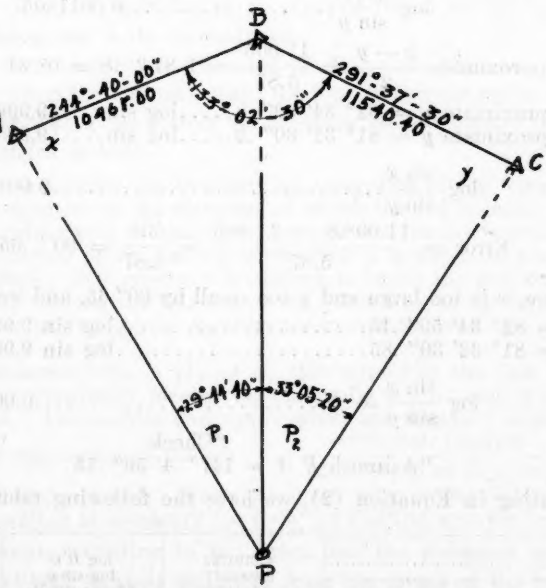
$$\frac{x+y}{2} = 360° - \frac{(B + P_1 + P_2)}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

$$P\,A = \frac{A\,B \sin x\, P_1}{\sin P_1} = \frac{B\,C \sin y}{\sin P_2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

$$\frac{\sin x}{\sin y} = \frac{B\,C \sin P_1}{A\,B \sin P_2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

$$(\log \sin x) - (\log \sin y) = \frac{\log B\,C \sin P_1}{A\,B \sin P_2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

"The solution consists in finding values for the angles, x and y, that will satisfy Equations (1) and (4).



SUGGESTED SOLUTION OF THE THREE-POINT PROBLEM.

"It is evident that if the correct value of either one of the angles, x or y, or the azimuth of either one of the lines, PA, PB, or PC, can be determined, the problem will be reduced to a solution of the triangles, PAB and PBC, and the two resulting values for length of the line, PB, will be the same.

"It is also apparent that any error in the assumed angle or azimuth will produce two values for length of the line, *PB*.

"In the graphical solution of the problem with the plane-table, by the method of approximation, the error of orientation is represented by a triangle of error at *P* when lines are drawn from *A*, *B*, and *C*.

"In the analytical solution the error of assumption is represented by the logarithmic difference between the first and second members of Equation (4), or the logarithm of the length of the side of the triangle of error drawn from the point, *B*.

"The solution consists in assuming values for the angles, x and y, to satisfy the conditions of Equation (1) and then reducing the error by approximation until the difference in log sines is equal to the second member of Equation (4).

"For example, suppose the known and observed parts to be as shown in the figure, we then have:

"From Equation (1):

$$\frac{x+y}{2} = 360° - \frac{(B + P_1 + P_2)}{2} = 82°\ 03'\ 45''$$

Difference in log sine for 2″ at 82° 03′ 45″ = 5.87

"From Equation (2):

$$\begin{array}{ll}
\log B\,C \ldots\ldots\ldots\ldots\ldots\ldots\ldots & 4.0622207 \\
\log \sin P_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots & 9.6955975 \\
\text{Co} \log A\,B \ldots\ldots\ldots\ldots\ldots\ldots\ldots & 5.9804268 \\
\text{Co} \log \sin P_2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots & 0.2628555 \\
\hline
\log \dfrac{\sin x}{\sin y} \ldots\ldots\ldots\ldots\ldots\ldots & 0.0011005
\end{array}$$

$$\text{"Approximate } \frac{x - y}{2} = \frac{11\ 005}{5.87} = 1\ 874''\ .8 = 0°\ 31'\ 14''\ .8$$

$$\begin{array}{ll}
\text{Approximate } x = 82°\ 34'\ 59''\ .8 \ldots\ldots \log \sin \ldots\ldots & 9.9963512.4 \\
\text{Approximate } y = 81°\ 31'\ 30''\ .2 \ldots\ldots \log \sin \ldots\ldots & 9.9952503.6 \\
\hline
\log \dfrac{\sin x}{\sin y} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots & 0.0011008.8
\end{array}$$

$$\text{Error} = \frac{11\ 008.8 - 11\ 005}{5.87} = \frac{3.8}{5.87} = 00''\ .65$$

"Therefore, x is too large and y too small by 00″.65, and we have,

$$\begin{array}{ll}
x = 82°\ 34'\ 59''\ .15 \ldots\ldots\ldots\ldots\ldots\ldots\ldots & \log \sin 9.9963511 \\
y = 81°\ 32'\ 30''\ .85 \ldots\ldots\ldots\ldots\ldots\ldots\ldots & \log \sin 9.9952506 \\
\hline
\log \dfrac{\sin x}{\sin y} = \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots & 0.0011005
\end{array}$$

"Check.

"Azimuth $P\,A$ = 147° .4′ 59″ .15

"Substituting in Equation (2), we have the following tabulation:

log *A B*	4.0194732	log *B C*	4.0622207
log sin *x*	9.9963511	log sin *y*	9.9952506
Co log sin P_1	0.3044025	Co log sin P_2	0.2628555
log *P A*	4.3203268	log *P A*	4.3203268
log *P B*	20 908.609	*P B*	20 908.609
Azimuth *P B* = 176° 59′ 39″ .15	....		....

"Generally approximate values of the angles, x and y, may be obtained from the traverse line, and the error is given by one approximation from the logarithmic tables.

"Rectangular co-ordinates of the point, P, are then computed, and the positions of the stadia stations adjusted between tie points, before the topography notes are plotted."

GENERAL DISCUSSION

A "CONSTANT ANGLE" STADIA SCALE

By J. R. Jahn,* Assoc. M. Am. Soc. C. E.

Calling attention to the fact that not much had been said about particular methods of transit and stadia work, in connection with Mr. Bayley's paper, Mr. Jahn took occasion to describe a logarithmic stadia board and a novel method of plotting stadia readings.

The usual stadia rod is divided in equal intervals. Mr. Jahn emphasized the fact that, when such a rod is held at distances of 1 500 or 2 000 ft., the widths of the graduations are no longer clearly apparent and a transitman may easily mistake the identification of a major division in concentrating on the smaller intervals, introducing an error of 100 or 200 ft. in an attempt to read stadia distances to the nearest foot.

On the constant angle rod discussed by Mr. Jahn the intervals increase logarithmically from the bottom toward the top. When the rod is held near-by the closer readings near the bottom are in range and, at greater distances, the interval is expanded so that,

"The graduation under the determining cross-wire of the telescope has the same width irrespective of the distance, at which the rod is held.

"A slight modification of the adjustment of the transit is necessary. The cross-hairs are moved up so that the lower wire is in the level plane when the bubble is centered. The eye-piece is shifted to bring the top wire back into the field without altering the relation of the lower wire to the level tube. The middle wire now has no special significance except for readings at more than 1 000 ft.

"In use the lower wire is placed on the 'target' at the foot of the scale which may be conveniently located on the stadia rod at, say, 4 ft. above the foot of the rod. The middle and upper wires are disposed along the stadia scale, the latter registering on the rod $\frac{i}{f} = \frac{\text{cross-hair interval}}{\text{focal length of objective}}$, of the distance from telescope to rod. Since the lower wire is parallel to the optical axis, only one setting is necessary for both rod reading and vertical angle."

It is important, according to Mr. Jahn, that the widths of scale-intervals be made proportional to their distances from the target at the bottom of the rod. This width he expressed as a formula, $\frac{d\,l}{d\,n} = K\,l$, in which, $d\,l$ represents the width of any interval, $d\,n$. Quoting Mr. Jahn further,

* Civ. Engr. with C. C. Kennedy, San Francisco, Calif.

"If K is made 0.02303 times the length or distance from the foot of the scale, then, by integration, the summation of graduations conforms to the formula, $n = \log_{10} l$, or $l = 10^n$, in which l is the distance, in feet, to the graduation, n. Thus, the constant-angle scale becomes a simple anti-logarithmic scale by which the scale intervals record the logarithms of actual lengths. Hence, the stadia wires register the logarithm of the rod intercept which is a function of the distance from rod to telescope.

"This property adds much to the value of the scale. Half intervals—that is, from lower to middle wire—are always just 0.301 less than the full intervals and recording becomes automatic. In other words, a middle wire reading of 0.927 is recorded 1.228 without hesitation or doubt [0.927 + 0.301 = 1.228].

"Furthermore, multiplications are performed by additions. The logarithm of the $\frac{f}{i}$ constant is seldom the log of 100 exactly, but is usually a small quantity, such as 0.002 to be added or 0.003 to be subtracted to give corrected rod readings.

"The horizontal component of any inclined reading is simply obtained by subtracting 0.000 — 2 log sin α, a small number which usually ranges between 0.000 to 0.100 and which may be inscribed in suitable manner on the vertical circle of the transit or alidade. The horizontal component can thus be written directly into the notes. Lest any doubt exist, no reduction from logs to feet is necessary, plotting being done with a scale similar to that of the logarithmic stadia rod, but reduced to the scale of the map. Thus, a map drawn to a scale of 1 : 31 230 would require a logarithmic scale approximately $\frac{1}{26}$th the length of the stadia rod, but with similarly disposed markings. It is convenient, however, to continue the graduations of the plotting scale to 1.301 which corresponds to a rod reading at 2 000 ft., the doubled half stadia interval of 10 ft. (which is the usual length of the stadia scale of this type).

"Having applied the horizontal reduction factor to the rod reading by subtracting the proper quantity, it is a simple matter to obtain the log of the difference of elevation between the transit and rod by adding the log of the tangent of the vertical angle to the logarithmic horizontal component and performing the final addition, or the subtraction, of the natural number corresponding thereto to the 'height of instrument'. A more convenient method involves the use of a nomograph of the **H** or **Z** type. A movable paper scale, equally divided, and movable along one leg of the nomograph, may be set to register thereon the elevation of the H. I. opposite a zero point on the nomograph. Elevations of all field points are then obtained directly at the point of crossing of the usual straight edge over the graduations of the nomograph."

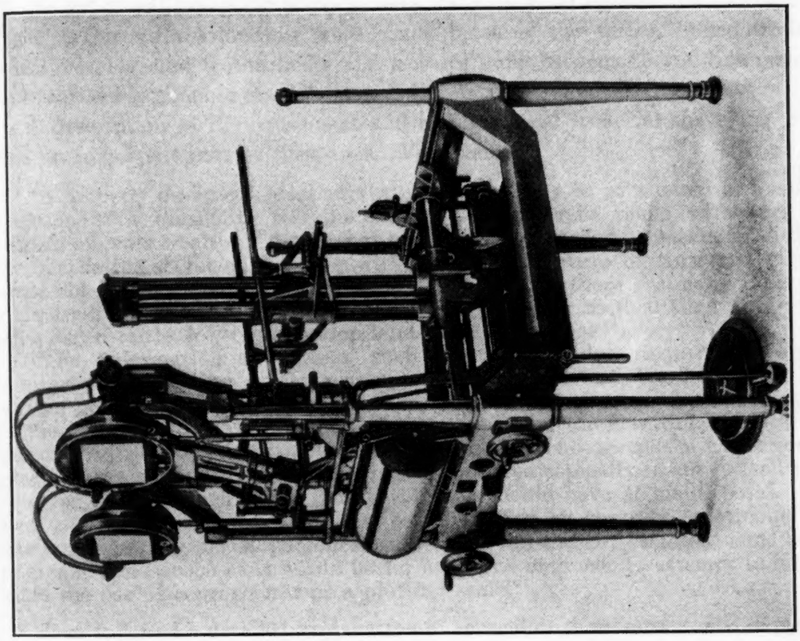
It is claimed for this equipment described by Mr. Jahn that it reduces the time usually required for observing, computing, and platting by at least one-half and increases the accuracy of the work. He urges surveyors to prepare the few necessary scales for use in topographic surveys of the magnitude described by Mr. Bayley.

THE AEROCARTOGRAPH METHOD OF PHOTO-TOPOGRAPHIC MAPPING

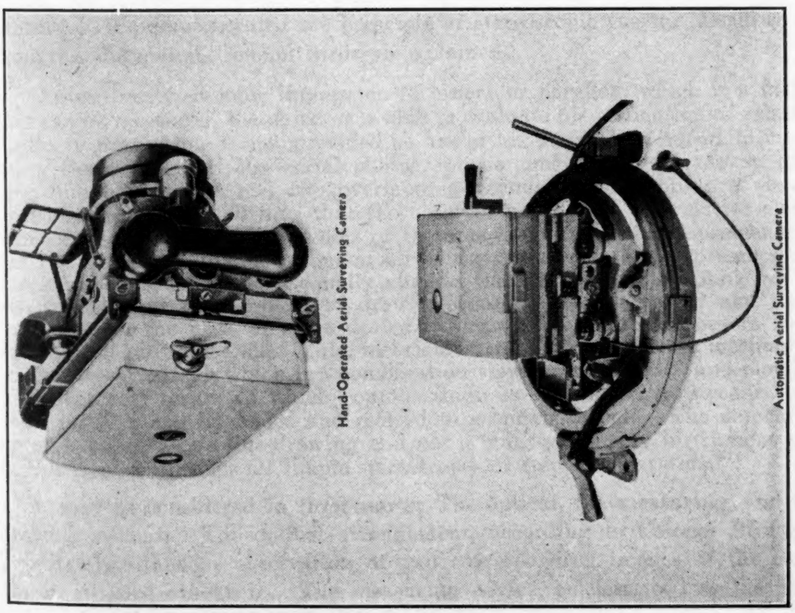
By C. H. Birdseye,* M. Am. Soc. C. E.

An especially valuable part of this paper was the author's statement defining common terms used in aerial photography. "A survey", said Colonel Birdseye,

* Pres., Aerotopograph Corporation of America, Washington, D. C.



THE HUGERSHOFF AEROCARTOGRAPH.



AERIAL SURVEYING CAMERAS.

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"is the act of gathering the physical data which is to be represented in graphic form". Careful distinctions were made between the terms, "uncontrolled aerial mosaics" and "controlled aerial mosaics", and between "aerial line maps" and "aerial topographic maps."

A description of the equipment and methods used in aerial surveying was also an important part of the paper. For example,

"A properly designed aerial surveying camera may be considered as a combination of a theodolite and plane-table. Photographs taken with such a camera afford a wealth of detail that could not be recorded by means of either the theodolite or the plane-table without an expenditure of time and effort often not justified. In fact, the speaker believes that there are many inaccessible areas which can be properly mapped by no other method than an aerial one; for instance, low-lying swamp lands or coastal areas.

"The examination and use of such photographic data require, however, a clear understanding of the principles of prospective and a knowledge of certain controlling factors, such as the position, elevation, and angle of tilt of the camera and the positions, and, in some cases, the elevations of certain ground control points. However, the use of such photographs without some practical mechanical means of viewing them stereoscopically is apt to lead the observer to miss much minor detail which he should have at his disposal. In fact, the speaker believes that it is as foolish for an observer to attempt to study overlapping aerial photographs without the use of, at least, some sort of simple stereoscope as it would be for a ground engineer to attempt to eliminate the use of a protractor or a plotting scale."

Then followed a detailed explanation of several of these mechanical devices. The principle of the measuring stereoscopes, including a so-called aerocartograph, was treated in detail. Numerous quotations from a paper* by Captain Hotine, R. E., demonstrated the principle of stereoscopic fusion. Leading on from this discussion, Colonel Birdseye explained,

"From the stereoscopic impression of binocular parallax, which is a factor that can be measured, the observer is able to evaluate his estimation of relative depths in measurable terms, provided he has at his disposal an adequate measuring stereoscope. If the aerial photograph be considered as made up of a large number of dots and two overlapping vertical photographs are viewed stereoscopically, it is obvious that the observer needs only the relative elevations of a sufficient number of dots or features to control his measurements in order to determine the elevations of all of the dots or features represented in the stereoscopic pair. It is equally obvious that if all the elevations can be determined, half the problem of drawing contour lines is solved and there remains only the need of some device to draw these contour lines to some definite and uniform scale. This, in brief, is accomplished by the mechanism of the aerocartograph which is a combination viewing, measuring, and plotting instrument, by means of which contour lines, as well as roads, streams, and other features can be traced and plotted to a common scale. The aerocartograph is essentially a line-drawing and not a point-measuring instrument and in this respect it excels all simple stereoscopes or stereocomparators."

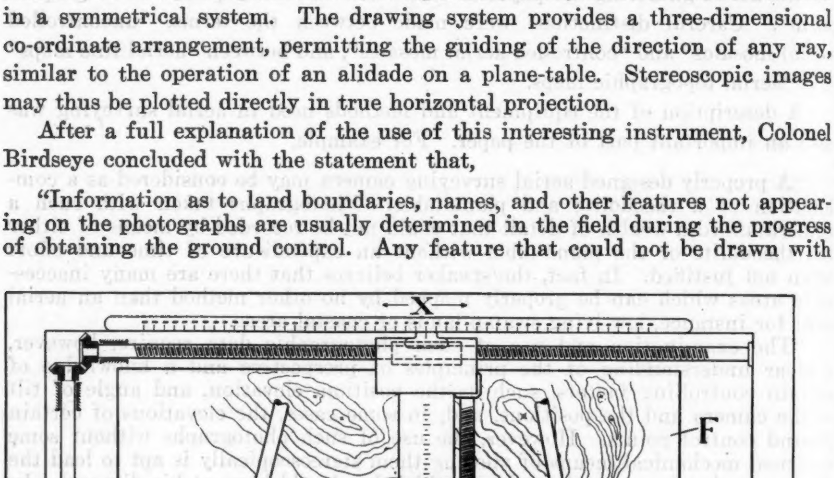
It may be considered in three parts: The optical, the measuring, and the drawing systems. The optical arrangement, according to Colonel Birdseye, permits simultaneous observation of two corresponding images of the same object, without eye-strain. The measuring device, he described as the com-

* *Professional Paper No. 4,* British Air Survey Committee.

bination of two photo-plane-tables, two photo-transits, or two inverted cameras in a symmetrical system. The drawing system provides a three-dimensional co-ordinate arrangement, permitting the guiding of the direction of any ray, similar to the operation of an alidade on a plane-table. Stereoscopic images may thus be plotted directly in true horizontal projection.

After a full explanation of the use of this interesting instrument, Colonel Birdseye concluded with the statement that,

"Information as to land boundaries, names, and other features not appearing on the photographs are usually determined in the field during the progress of obtaining the ground control. Any feature that could not be drawn with



VERTICAL VIEW OF AEROCARTOGRAPH, SHOWING MAP IN PROCESS OF CONSTRUCTION.

the aerocartograph because of natural obstructions or obscure parts of photographs are added by ground survey methods, but such operations are usually of very limited extent. * * * * * * * *

"The economic value of good maps has long been apparent to the map producer. It is believed that the value of maps constructed according to proper standards is being more and more recognized by the profession, by those in authority in the development of projects involving the use of our natural resources, and by the general public. This recognition will result in an increased demand for better maps, and it is believed that aerial photographic methods will serve, more and more, as an important aid to the map producer in meeting this demand."

GENERAL DISCUSSION

DeWitt L. Reaburn, M. Am. Soc. C. E., requested information as to the time required for making a finished map with the methods described by Colonel Birdseye. To this, the author replied that there were so many factors to consider that a definite statement was difficult. For example, he pointed out that, with a properly equipped camera, the progress may be as much as 100 or even 200 sq. miles per day. On the other hand, he said, there may only be "two or three good flying days during an entire month."

With the instrument he is now using, Colonel Birdseye explained that it was not possible to produce maps with a scale smaller than 1 in 24 000; that is, 2 000 ft. to the inch. With this scale and with 20-ft. contours, the aerocartograph can complete the drawing of 1 sq. mile in 1½ days, or about 12 working hours. By ordinary plane-table methods, surveying the same territory, Colonel Birdseye expressed the belief that the territory covered would not be more than ½ sq. mile.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

REFLECTIONS ON THE STATUS OF THE ENGINEER

ADDRESS AT THE ANNUAL CONVENTION AT CLEVELAND, OHIO, JULY 9, 1930

By J. F. Coleman,* President, Am. Soc. C. E.

For years there was a constant cry to the effect that the engineer did not receive the recognition which he deserved. Lately, the same cry is heard although less frequently. In the past there was much to justify such complaint, and even now there is some excuse for it.

It seems probable, however, that the engineer himself is in great degree responsible for such a state of affairs in that he has been until recent times almost inarticulate in the councils of men; and, at this date, while by no means silent, is heard far less frequently than he should be. It is astonishing how few laymen know what an engineer is and how he accomplishes the tasks allotted to him. For that situation, the engineer is more to blame than the layman; and it would be distinctly to his advantage to have others know more of him and his work than they now do. It may therefore profit the engineer to analyze the matter and to take steps to overcome this disability.

Leaders of the profession have preached for years on the desirability of its members taking part in civic and other public affairs, and as time has passed, these preachings have borne some fruit. The engineer is to-day more prominently in the public eye than he has ever been before, and without doubt this improved position has been brought about at least in part by those engineers who have given of their time and their talents to public affairs.

It has been stated that in the 1929 edition of "Who's Who in America", there are 2 858 engineers and architects, among which number there are or have been 10 Governors, 13 Members of Congress, 2 Members of the Cabinet, and the President of the United States. There are also engineer presidents and a number of other engineer executives in large and important corporations. The profession is represented in chambers of commerce and other similar organizations and, on the whole, enjoys a better position than it formerly did.

* (J. F. Coleman Eng. Co.) New Orleans, La.

Notwithstanding these facts, there appears to be an almost abysmal ignorance on the part of the layman as to what an engineer is and does. He knows that the engineer builds bridges, but he has no conception as to the designing of such structures, which must precede the construction. He knows that the engineer builds subway systems, railways, manufacturing plants, and water-works, but his information does not extend to any conception of what the engineer must know and do to plan such works in an orderly and intelligent manner before the actual construction may begin.

From his contacts with the doctor in the sick room or with the lawyer in the court room, he has some idea as to how the members of those honorable professions function. He is practically never present, however, at any stage of the work of the engineer, except that stage in which the construction is in progress and manned almost wholly by laborers skilled and unskilled, and, therefore, he has no visual evidence of the guiding hand and brain of the engineer who planned the structure and who directs the details of its execution.

It is a fact that in his daily life the layman is in more frequent contact with the products of the brain and skill of the engineer than with those of any other calling, but he takes these things as much for granted as he does the sunshine and the rain. The water in which he takes his bath; the electricity and gas which furnish light and fuel; the transportation agencies which bring his food to his door; the street railway, the automobile, or the railroad, which conveys him to his office; the office building itself, with its equipment of elevators, heating system, etc. All these are the products of engineering achievement as are countless other items of daily use, such as the telephone, the telegraph, the radio, and the airplane; but so far as conscious thought goes, to the average layman they "just happened", and he uses them without thought as to how they happened.

The World War was often spoken of and written about as an "Engineers' War", although it would be hard to find in any such spoken or written statement any information as to what made it such. As engineers, we have knowledge of those factors which caused it to be so designated. The various agencies whereby large bodies of men and large volumes of supplies were transported and handled; the various means of rapid communication, such as the telephone, the telegraph, and the radio; the submarine and the airplane; guns both large and small; and explosives of all classes and for all purposes. All these and countless other items important in war represent engineering achievement.

By reason of the prominence given to the name, "Engineer", during the World War, this designation attracted the favorable attention of people of all classes, some of whom showed a desire to capitalize on the public favor by calling themselves engineers or by attaching the word, engineer, to the name of their trade or calling. We thus had Forgery Prevention Engineers, Milk Engineers, Feed Plant Engineers, Socio-Religious Engineers, Display Engineers, etc., *ad infinitum*.

While the profession may have suffered to some extent from such practice on the part of these unauthorized persons, it is certain that it was extensively advertised, and it seems fair to presume that it gained as much as it lost.

Without doubt, each profession suffers from some disadvantage by comparison with others. Among those from which the Engineering Profession suffers and must continue to suffer is the fact that so large a percentage of its members are salaried employees. Whatever his capacity, the salaried employee must attain a position of some importance in the organization to which he is attached before he can expect to receive much consideration outside that organization. In his contacts with other men, he is handicapped by being a subordinate. He cannot be the master of his own time, nor can he be as free and independent in the expression of his views as are those who both work and speak for themselves. Under such circumstances, he is far less likely to seek or to accept opportunity for public expression of his views than is the man of any calling who is not on anybody's payroll.

In the performance of his professional work, the engineer is far more accustomed to the quietude of his office, his drafting room, or his home, where he analyzes data, studies reports, and the like, than he is to making public talks or speeches. His whole training and experience makes him somewhat hesitant in presenting his ideas until he feels that he has fully matured them and is in possession of all available facts.

These considerations being understood, it is easy to see why the engineer has been more or less inarticulate, and it is surprising that he has progressed as far as he has into the public consciousness. No man or set of men will ever get very far or make much progress if satisfied with the *status quo*. A healthy state of dissatisfaction is necessary to progress in any branch of human endeavor. The somewhat remarkable progress of the Art of Engineering bears testimony to the fact that the engineer has that healthy state of dissatisfaction.

While no such progress has been made in the status of the engineer as in the state of the art, it does not indicate a satisfied state of mind with respect to the status. It is rather an indication that the engineer has a more lively interest in his work than in himself.

In an attempt to bring about some improvement in the status of the engineer, the American Society of Civil Engineers has within the past few months adopted a program for the broadening of its activities so as to reach into fields not hitherto covered by it. Committees have been appointed and have begun to attack the problems assigned to them, so that work is now in progress in a number of ways looking toward the desired end. The task is not an easy one, nor is it so definite in its nature as are many of the more technical ones which the engineer must approach. Also, it is a task which cannot wholly be accomplished by any number of committees. The task of the committees is rather one of determining upon the best means to be adopted to achieve results.

Each individual member of the profession has a duty to perform to attain the hoped-for ends. Each man may be and should be a "focus of infection" to his friends and neighbors and to others with whom he comes in contact. He should avail himself of reasonable opportunities to convey to the minds of others those things which the layman should know of the work of the engineer.

To be most effective, there should be avoidance of anything like boosting or advertising of individuals in the interesting bits of information presented.

There are many most interesting facts which could be conveyed to laymen in ordinary conversation, or in brief talks, which would tend to create in the minds of the hearers a better understanding of the mental equipment necessary to the engineer, and opportunities for so doing are plentiful without the necessity for creating them.

Those who avail themselves of such opportunities are almost certain to reap individual reward even though they do not seem to seek it. Every man who conveys an interesting bit of knowledge to a fellow man attracts some measure of attention and respect from that fellow man, and such respect and recognition by one's fellows in sufficient number will assuredly result to advantage in a material way.

The engineer is as anxious for opportunity to improve his earning capacity as any other man. He is as ready appreciatively to accept added income and better surroundings as the next one. It has been charged that he is not as ready and willing to assume civic duties which do not carry financial returns. It is more likely, however, that he is not tendered such duties so frequently as are the representatives of other fields of endeavor, and this for reasons which have hereinbefore been noted. Such assignments are not infrequently the means whereby a man may disclose his capacities, and they sometimes result in his advancement.

The engineer is apparently often overlooked in the community in which he resides, and he has always seemed less ready to push himself forward and to express his thoughts than are men in other avenues of life. It is only when a man has achieved some degree of prominence that his words are likely to be heard with respect and attention; and the engineer being in the mass modest, or one might say super-modest, appears to hesitate about taking issue on general subjects with other men who either have or assume positions of greater prominence.

Possibly local sections of the National Engineering Societies and local engineering societies may aid in bettering things. They may be able to foster a proper public spirit among their members, and no doubt there are opportunities for them to put forward men for civic and other public work who will reflect credit upon themselves and upon the profession. The engineering societies and individual engineers should hold themselves in readiness to grasp opportunities of this nature as may come to them.

The American Society of Civil Engineers has now definitely set its feet in this direction and without doubt will continue in that course. With a reasonable amount of well-directed aid on the part of the individual members, it seems safe to predict good results. It is to be hoped, however, that as a profession and as a Society, we shall never become wholly "satisfied", as in that would lie inertia, atrophy, and, ultimately, dissolution.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

HIGHWAYS AS ELEMENTS OF TRANSPORTATION

By Fred Lavis,* M. Am. Soc. C. E.

Synopsis

The paper shows that, for handling large volumes of traffic through congested areas, highways should be designed for the main purpose of transportation rather than as adjuncts of the abutting properties. It is pointed out that where congestion of existing city streets interferes with the reasonably free passage of vehicles and pedestrians, highways of the type described are necessary, not only to expedite through traffic, but also to restore such streets to their normal uses.

Next, there follows a brief description of the through trunk highway recently built by the State of New Jersey through Jersey City and Newark; discusses the factors which influenced its location; and develops a general economic theory which governs the relation between cost of construction and the operating costs of vehicles using highways of this character. This theory is based on the principles of Wellington's "Economic Theory of Railway Location."

Historical Development

Up to within very recent times, highways in large part have been developments of the old trails made by man before the eras of wheeled vehicles, or they have been laid out as means of reaching real property. Their design and construction as elements of transportation are very recent and this is required of course, by the intensive development and use of motor vehicles.

Older Highways.—Economic factors developed by the use of hard surfaces for highways were probably first recognized by Macadam and others at the end of the Eighteenth and the beginning of the Nineteenth Century. Experiments to determine the tractive effort required for haulage over various types of road surfaces were made by several investigators between 1840 and 1850.

Note.—Written discussion on this paper will be closed in November, 1930, *Proceedings*.

* Pres., International Rys. of Central America, New York, N. Y.

In spite of the fact, however, that the economic relation between road surfaces, and the tractive effort required to haul loads, was recognized nearly a century ago, by a few competent engineers—men who recognized the principle that true engineering and economics are indissolubly linked together—there has not been, at least until recently, any attempt to actually evaluate the effect of the different elements of the location and types of construction on the cost of operation of the vehicles using the highways.

Motor Vehicles and Modern Highways.—The introduction of the motor vehicle has had a very notable effect on the construction of highway pavements, and has resulted also in certain improvements in alignment, rates of gradient, rise and fall, etc. Expenditures for these purposes have been enormous and are continuing at a rate which is almost staggering. Changes in the locations of existing highways, however, are quite definitely limited by long established conditions; such roads are an integral part of individual, as well as municipal and State, property rights; and any contemplated change must take these rights into consideration.

There has developed, however, in recent years a demand for a new type of road—a road more nearly analogous to the railroad; laid out on an entirely new right of way and designed and built primarily to meet a demand for transportation. Such roads must necessarily be designed from the point of view of their users rather than that of owners of abutting property.

The design and construction of this more modern road are based primarily on the needs of a large volume of intensely mobile traffic which desires to move from one place to another with the least resistance compatible with the physical conditions of the territory through which it must pass. It is a development of modern transportation which must be recognized and provided for.

Transportation a Factor.—The last century has shown the ever-increasing necessity and value of transportation as a factor in the development of commerce and as perhaps the most important and dominant factor in modern civilization.

The railway was and probably still is the most important element in this modern transportation development, other factors being steamships, motor vehicles, air craft, the telegraph, the telephone, and the radio. Modern business requires that railway traffic should not be delayed by stops at grade crossings, and the modern efficient railway is designed with alignment, rates of gradient, rise and fall, multiple tracks and every adjunct necessary to permit the free rapid movement of its traffic.

These same conditions apply in degree to the type of highway herein described. The volume of railroad business handled has permitted large expenditures for improvements in the road-bed and for the removal of resistances and delays to the passage of trains, so that freight and passengers are not only moved in quantity at high rates of speed, but also at low cost. Highways must be developed to permit the same safe uninterrupted flow of heavy traffic as now obtains on a modern trunk-line railway, and the cost of operating these motor vehicles becomes an economic factor of importance.

These results on the railways have been achieved by a careful consideration of the economic values involved, the relation between location, costs of construction, and costs of operation. A similar era is impending in connection with the construction of highways. The following discussion follows the principle of the "Economic Theory of Railway Location," developed by the late Arthur M. Wellington, M. Am. Soc. C. E.

Need of Special Highways.—It is not only necessary to provide smooth pavements as a matter of comfort and convenience, but it is more than ever necessary where the traffic demand is large, to provide for the free uninterrupted flow of the through traffic without interference with local uses.

In open country the ordinary types of modern highways can and do carry this traffic with reasonable efficiency and dispatch, but in the vicinity of large cities, and in these cities themselves, special provision for through traffic which there reaches large proportions, must be made so that the ordinary city streets may serve local needs and abutting property.

It is with the design of these special highways that this paper is principally concerned and its object is to emphasize the need of making such design and considering such highways on the basis of their efficiency as elements of transportation. These are trunk highways, located on their own rights of way and designed primarily for the economic movement of a large volume of traffic through a given area or between given points.

A Modern Highway

Route 25, New Jersey.—The nature of the problem and suggested methods of solution can best be indicated perhaps by a description of the design and construction of the arterial highway between the Hudson River and Elizabeth, known as Route 25 of the New Jersey State Highways. This is also the eastern end of the transcontinental Lincoln Highway. It has a roadway 50 ft. wide, sufficient for five lanes of traffic, maximum gradients of 3.5%, and curves with a minimum radius of 1 000 ft.

At its eastern end (Fig. 1) it connects with the Holland Vehicular Tunnel under the Hudson River. Thence, it passes through Jersey City and thus over the trap dike which divides the Hudson River from the valley of the Lower Hackensack and Passaic Rivers (both being navigable streams), crosses this valley in the lower part of the Town of Kearny, then by-passes the City of Newark, skirting the edge of the uplands and meadows, and, finally, crosses the meadows to the City of Elizabeth, beyond which it connects with the main State highway routes leading toward the South.

The first 8 miles of this route are without crossings of other highways at grade, thus providing for the free, uninterrupted flow of traffic through the highly congested areas of Jersey City and Newark. Provision is made, of course, for connections with important highways by ramps, thus avoiding all cross-traffic currents. This 8-mile section was estimated to cost \$25 000 000, and, in connection with its location and design, a careful study was made of the governing economic factors.

General Considerations Affecting Design.—The so-called Metropolitan Area of Northern New Jersey comprises the Cities of Bayonne, Jersey City, Elizabeth, Newark, the Oranges, Montclair, Paterson, and Passaic, and other

smaller communities. From a traffic point of view it constitutes a continuous, densely populated, barrier between the City of New York and the South and West of the United States. It is also in itself both a source and a destination of a large volume of traffic.

The Western States may be reached from New York City by traveling northward along the Hudson River Valley to Albany, N. Y., and then striking west, but the natural route is to cross the Hudson from New York City to New Jersey and continue westerly. Such traffic must then necessarily pass through this congested zone, as must, of course, all traffic destined to the Jersey Coast resorts, to Trenton, Philadelphia, Baltimore, Washington, and the South.

The construction of the Holland Vehicular Tunnel concentrated a large part of this trans-fluvial and transcontinental traffic at the lower end of New York and at the most congested area of Jersey City. West of the Hackensack and Passaic River Valleys the traffic again had to find its way by many twists and turns through the overburdened streets of the Cities of Newark and Elizabeth.

The primary cause for consideration of this highway was the need of relief for these overburdened streets in Jersey City, Newark, and Elizabeth, traffic on which had become so congested that neither citizens who desired to shop, shopkeepers who desired to sell, or manufacturers who desired to move materials, could do business except at great expense and delay.

Traffic counts at fourteen points in the City of Newark showed increases for the decade, 1912 to 1922, ranging from 64% to 1 183%, the average being 340 per cent. The increase in traffic on the main State highway at Rahway just south of Elizabeth between 1921 and 1926 was from about 12 000 to 44 000 vehicles for a 24-hour period, that is, nearly fourfold in five years. Other counts indicated similar increases throughout the region.

Most of Northern New Jersey is not only a manufacturing district of great importance, but one so conveniently accessible to the Port and City of New York and to railroad and highway transportation, to a great labor market, and to financial centers, that it is destined to continue to grow, and probably grow very rapidly, in importance, both as a manufacturing, retail business, and residential district.

This highway, therefore, has two important purposes: It will permit the through traffic to cross this congested area, leaving existing city streets and highways free for their normal local uses; and it will permit easy, rapid access between various points in this area, and between all these points and New York City. Further, the great airport built by the City of Newark, although separated from Broadway and the down-town business district of New York City by 10 miles of congested and densely built-up territory, can be reached over Route 25 within 20 to 30 min.

Transportation to Be Provided

Amount of Traffic.—In developing the economic factors to determine values of the physical characteristics of this highway, curves were platted showing the actual existing traffic, its increase during the previous few years, and its probable future increase. These estimates not only were based on counts

of traffic, motor vehicle registrations, etc., but they were checked by traffic statistics of other carriers, such as the ferries, railways, and the Holland Tunnel, by charts of the growth of population and use of motor vehicles, by similar statistics of other areas, etc., and by the peculiarities of this particular district.

The application of these data to the future is necessarily largely a matter of judgment, but, reviewing all the information assembled, it seemed entirely probable that the highway would be used to capacity very shortly after its completion. It then remained to determine its capacity, or the number of vehicles which would use it, in order to obtain a basis for the determination of the economic values.

Highway Capacity.—The highway is built with a 50-ft. roadway and is expected to carry four lanes of continuously moving traffic with one 10-ft. lane to spare.

Observations taken (which it is believed check with studies by others) indicated that with freely moving traffic the largest number of vehicles pass a given point when moving at the rate of about 15 to 20 miles per hour. On this assumption four lanes would give a maximum capacity of 5 455 vehicles per hour. It is well known, of course, that traffic not only varies throughout the hours of the day, but also on different days of the week and at different seasons.

The number of vehicles which can use any given highway at times of maximum demand also depends on the proportion of trucks and busses or other types of heavy vehicles to lighter or non-commercial types. (It is not expected that horse-drawn vehicles will use this highway.)

On Route 25, it was necessary to consider heavy peak loads at certain times, as for traffic to and from the New Jersey shore resorts over week-ends, to football games at Princeton, etc. The computation of the daily traffic, therefore, was based on the assumptions in Table 1; and it will be noted that, at times of peak traffic in one direction, it was assumed that the traffic on the other two lanes would be at half capacity.

Based on these assumptions, therefore, and on the desire to be quite conservative in all estimates, it was assumed that the maximum traffic would be 3 600 vehicles per hour, and the average, 54 000 per day, with a total of 18 360 000 per annum.

Economic Factors

Cost of Delays.—One factor of great importance economically is that of the cost of delays, due to interruptions at grade crossings with other highways, etc. This factor, of course, has not been considered in railway practice because of the obvious impracticability of stopping trains for these causes, although consideration has been given to the costs of delays of trains held at sidings for the passage of opposing trains on single-track lines, and the effect of this on economies of operation.

Necessarily, the cost of delays must be measured, in the first instance, by a computation of the time lost by each vehicle and the total time lost by all

vehicles delayed. It is obvious, also, that the delays at grade crossings will reach a maximum at times of peak loads, and be almost nil at hours of minimum traffic. At a drawbridge the amount of delays will be the time lost by the actual number of vehicles held up during openings.

TABLE 1.—ESTIMATED DAILY TRAFFIC, ROUTE 25.

Hour.	Percentage of maximum traffic.	CARS PER HOUR PER LINE.		Total cars per hour.
		1 and 2.	3 and 4.	
12:00–1:00 A. M..........	70	840	420	2 520
1:00–2:00 A. M..........	40	480	240	1 440
2:00–3:00 A. M..........	10	120	60	360
3:00–4:00 A. M..........	10	120	60	360
4:00–5:00 A. M..........	10	120	60	360
5:00–6:00 A. M..........	10	120	60	360
6:00–7:00 A. M..........	10	120	60	360
7:00–8:00 A. M..........	20	240	120	720
8:00–9:00 A. M..........	30	360	180	1 080
9:00–10:00 A. M..........	40	480	240	1 440
10:00–11:00 A. M..........	50	600	300	1 800
11:00–12:00 A. M..........	60	720	360	2 160
12:00–1:00 P. M..........	70	840	420	2 520
1:00–2:00 P. M..........	80	960	480	2 880
2:00–3:00 P. M..........	90	1 080	540	3 240
3:00–4:00 P. M..........	100	1 200	600	3 600
4:00–5:00 P. M..........	100	1 200	600	3 600
5:00–6:00 P. M..........	100	1 200	600	3 600
6:00–7:00 P. M..........	100	1 200	600	3 600
7:00–8:00 P. M..........	100	1 200	600	3 600
8:00–9:00 P. M..........	100	1 200	600	3 600
9:00–10:00 P. M..........	100	1 200	600	3 600
10:00–11:00 P. M..........	100	1 200	600	3 600
11:00–12:00 P. M..........	100	1 200	600	3 600
Total daily capacity (cars)..........	....	....	....	54 000

Due consideration also must be given to the loss involved by reason of the decreased use of the highway as a whole, or at least on the section affected by the delays. If, for instance, the normal capacity of 20 cars per lane per min. is reduced by reason of delays to 15 cars, the traffic capacity of the highway at times of peak demand is reduced by 25 per cent. Obviously, also, the cost of the delays must be calculated for the number of cars which can actually use the highway and not for the number which could use it if the traffic flow were free and uninterrupted.

In the case of a highway where the traffic demand is equal to the capacity of the highway any reduction in this capacity must be evaluated on the basis of the cost of the facilities so provided and not utilized, or on the basis of the cost of other facilities necessary to take care of the traffic not accommodated. On Route 25, it was estimated that if the traffic were delayed at a crossing at grade with another highway under such conditions that the traffic on the main highway could flow continuously for 3 min. and be interrupted for 1 min., the daily loss to the traffic on the two highways would be:

On the main highway.............	14 532	car-min. per day
On the cross street..............	9 000	car-min. per day
Total	23 532	car-min. per day

To be conservative and in view of all the conditions it was further assumed that the loss per annum would be at the rate of 7 000 000 car-min.

Assuming (on a basis to be developed) that a car-minute may be valued at 2 cents the yearly loss at such a crossing will be $140 000. This, capitalized at 6%, represents a value of $2 333 000. In addition, it was estimated that by reason of these delays, the traffic-carrying capacity was reduced by 12.1 per cent. If the cost of the highway be assumed to be $22 000 000, then this loss in efficiency can be calculated to be $2 662 000. The total amount, therefore, which might profitably be spent to eliminate a single isolated crossing under the conditions assumed would then be approximately $5 000 000.

Of course, if there were a group of crossings more or less close together, and if the traffic were properly regulated by a system of adequately controlled and synchronized signals, the economic loss due to the group would be only slightly greater than that due to one crossing only. It was assumed that the amount which could profitably be spent to eliminate a group of from two to five crossings would be as follows:

One crossing	$5 000 000
Two crossings	5 300 000
Three crossings	5 600 000
Four crossings	5 840 000
Five crossings	6 080 000

It will be obvious, of course, that this aspect, as in fact all other aspects, of this problem, must be studied carefully from its own individual point of view, taking into consideration the effect of all local conditions. The mathematical computations must necessarily be made and interpreted with a great deal of judgment.

Drawbridge Delays.—Route 25 had to cross the Hackensack and Passaic Rivers. The crossings of these rivers were studied by the writer for tunnels, and for bridges with movable spans having 35 ft. clear head-room above mean high water as required by the War Department. (Later, studies were made per fixed spans with 135 ft. clear head-room.)

The data on which calculations of the costs of delays at the drawbridges were based, were taken from the draw-tenders' records at adjacent bridges on the two rivers. These showed the number of openings, duration of each, and time of day. In using these data account was taken of the fact that the proposed bridges on the new line would have 35 ft. clear head-room above high water, when closed, instead of the 10 or 12 ft. of the existing bridges, and this requires fewer openings, as much of the river traffic could then pass without affecting the drawbridges. Future increases in the river traffic were estimated and allowance made therefor.

As in the case of grade crossings with other highways, two factors were taken into consideration: Actual losses due to delays of traffic, and losses due to decrease in capacity of the highway. In estimating the actual losses to vehicles, the average number of openings of the bridge for each hour of the day was applied to the number of cars estimated to use the highway during that hour, and the loss in car-minutes thus calculated.

These two items for the two bridges were evaluated as follows:

For traffic delays, 32 470 000 car-min. at 2.0 cents per min., $649 400 capitalized at 6%.........	$10 823 000
Loss of efficiency, 6.33% of $22 000 000.........	1 392 600
Total capitalized value of delays due to openings of the drawbridges.........	$12 215 600

Value of a Car-Minute.—Reference has been made to the value of a "car-minute", that is, the costs of delays per car per minute. Of course, this differs for each type of vehicle, but the general methods of estimating it are the same, except that for commercial vehicles there are more definite charges for delays which prevent the operation of this part of the owner's plant. It will be realized that a motor vehicle used for commercial purposes is just as much part of a manufacturing plant or selling organization, as a machine in a factory, or an employee in a store. If idle when it might be working, it represents an item of loss.

For commercial motor vehicles, therefore, the stoppage is an element of expense. For non-commercial vehicles, however, the loss is not so clear, as, at first thought, it would seem to be a matter of comparative indifference whether a passenger arrived at his destination 2 or 3 min. earlier or later.

An experience with a taxicab in a traffic jam is, however, an illuminating example of what these delays actually cost; and if, for instance, one happened to be in a line waiting to cross the Hudson River by ferry, one would usually be willing to pay something extra for the privilege of moving up to the head of the line and avoiding the delay. The extra payments made to agents for theater tickets to avoid waiting at the box office, the rush for the first car of a train, etc., are all evidences of an innate realization of the costs of delays. The value of this economic factor of loss due to small delays has been doubted by many, not only laymen but engineers; nevertheless, it is a factor which must be given consideration.

These delays, of course, affect only some of the costs of operation. For purposes of analysis the average costs for a 3-ton truck for one year in the vicinity of New York were used for these estimates approximately as given in Table 2.

TABLE 2.—Yearly Expense of Running 3-Ton Truck, New York City.

Item.	Classification.	Amount.
1	Interest on $3 000 at 6%..........	$ 180
2	Depreciation, average life of 5 years..........	600
3	Insurance..........	300
4	Driver's wages, 300 days at $5.60..........	1 680
5	License fee..........	20
6	Gasoline, 6 miles per gal. at 25 cents..........	500
7	Lubricants..........	75
8	Tires..........	270
9	Repairs and maintenance..........	360
10	Miscellaneous..........	200
11	Overhead, 20%..........	837
Total..........	....	$5 022

If a vehicle be delayed or stopped, certain expenses continue, such as Items Nos. 1, 2, 3, 4, 5, and 11. If during the delay the engine continues to run,

there is also, of course, a certain consumption of gasoline and lubricants, but this will be small and has been omitted in the calculations.

Using the data in Table 2, and making similar allowances for light commercial vehicles and passenger cars used for commercial purposes, the following values for the cost of delaying a car for 1 min. were established:

For trucks	2.3 cents per car-min.
For light commercial vehicles.......	2.1 cents per car-min.
For non-commercial vehicles........	1.0 cent per car-min.

It was further assumed that for this particular highway, the economic values should be based on the assumption that the traffic would be divided in the following proportions:

	Percentage.
Trucks	50
Light commercial vehicles....................	25
Non-commercial vehicles......................	25

The average value of a car-minute (delays per car per minute), therefore, was assumed to be 2.00 cents.

Savings in Distance.—The economic effect of distance may be considered under two headings: Time, and operating cost. The value of time has already been studied in connection with delays at grade crossings and at drawbridges. It remains, therefore, to consider differences in operating costs due to increases or decreases of distance. These will be considered without reference to other elements, such as curvature, rise and fall, rates of gradient, or pavement surfaces. The cost of additional distance is affected, of course, quite appreciably by differences in types of pavement, but it can be assumed that the pavement will be of the same character on any two locations of the same route which may be under comparison, and that for highways of this type the pavement will generally be a smooth hard surface.

In computing operating costs not only are there several different classes of highways to be considered, but an almost bewildering variety of types and weights of cars, and almost as many degrees of ability in their operation and in conditions of efficiency. The determination of a cost unit, therefore, is not a matter of scientific accuracy. It has seemed possible, however, to develop average figures which are sufficiently accurate for the purpose in view, that is, the comparative operating costs of two locations on the same route on which the traffic is to be the same.

In the estimates for Route 25 a comparison of operating costs obtained from various sources indicated that for the items affected, namely, depreciation; fuel; lubricants; tires; repairs; and miscellaneous (see Table 2), the following is probably a fair average cost of operation for the traffic expected to use this highway:

	Cents.
50% heavy trucks at 15 cents per mile........	7.5
25% light trucks and buses at 10 cents per mile	2.5
25% non-commercial at 6 cents per mile......	1.5
Average per mile................................	11.5
Equivalent to	2.18 per 1 000 ft.

Assuming that 18 360 000 vehicles will use the highway annually, the decreased operating costs, due to a shortened distance of 1 000 ft., will be $400 248 per annum, which, capitalized at 6%, indicates that the sum of $6 670 000 might profitably be spent to save this distance.

Rise and Fall.—Results of experiments indicate that, because of certain deficiencies in the design of engines of motor vehicles, the cost of operation on moderate ascending gradients is no greater than on the level. While this may be true, it is perhaps reasonable to assume that such deficiencies may be corrected, and that the additional cost of operation on ascending gradients is that required to raise the load to a given height.

Experiments made at Iowa State College* indicated that the work done by a motor vehicle to raise a weight to a height of 1 ft. is approximately equal to the force required to propel the same weight over a level distance of 50 ft. on good pavements.

Taking the data previously used, it will be found that for the average car on good pavements, the cost of producing power only, that is, fuel and oil, is approximately 3.22 cts. per mile, or about 0.03 cts. for 50 ft. Therefore, this may be assumed to be the cost of raising the average car (on this highway) 1 ft.

It is realized, of course, that on highways as on railways, momentum will overcome certain smaller elevations without expenditure of power. It is considered further that the expenditure of power on light rates of gradient within certain limits of height may be negligible, but no information seems available to determine the limits within which this may apply. Arbitrary assumptions, therefore, were made in estimating operating costs due to rise and fall, but all these were believed to be conservative and to represent smaller, rather than greater, actual operating costs for this factor.

For a traffic of 18 360 000 cars per annum, the cost of 1 ft. of rise and fall at the rate of 0.03 cents per car is approximately $5 508, which, capitalized at 6%, represents a value of $91 800 as the amount which might profitably be expended to eliminate 1 ft. of rise and fall.

However, after due consideration of all the factors involved—and it may be noted again that this is a purely arbitrary assumption—it was decided on this Route 25 that values for rise and fall should only be applied to gradients of 2% or more (the maximum is 3.5%) and be varied for total lengths of continuous rise, as follows:

For total elevations of:	Profitable expenditure to reduce by 1 ft.
15 ft., or less	None.
18 to 20 ft.	$30 600
20 to 25 ft.	$61 200
25 ft., or more	$91 800

Curvature.—On Route 25, the minimum radius of curvature was fixed at 1 000 ft. It was considered that, for the proposed conditions, that is, a well-paved-non-skid (granite block) pavement, 50 ft. wide, curvature would have little or no effect on operating costs or capacity of the highway to carry traffic.

* "The Economics of Highway Grades," by T. R. Agg, M. Am. Soc. C. E., *Bulletin 65*, Eng. Experiment Station, Iowa State Coll., Ames, Iowa, February 28, 1923.

However, for highways with sharp curvature, where visibility is reduced, or conditions exist which slow up traffic, there will be costs due to delays and, in cases of maximum traffic demand, costs due to a decreased capacity of the highway to carry traffic.

Evidently, a certain amount of power is required to change the direction of a moving vehicle, but, so far as the writer knows, no experiments have been made to determine how much this may be for highway motor vehicles. Probably a reasonable estimate of the costs of delays, accident risk, and loss of capacity of any given highway would be a fair estimate of the cost of curvature or of the value of its removal. For the highway under consideration no account was taken of additional operating costs due to curvature.

Design of Route 25

Application of Economic Theory to Actual Location.—In a general way two sets of factors determine the location of a new highway of the character herein described:

> *First.*—The physical characteristics of the terrain and the need of reaching certain points for the benefit of the traffic and of the territory to be served; and
>
> *Second.*—The economic factors affecting the operation of vehicles over the route.

The original legislative act authorizing the construction of this highway proposed it as an extension of old Route 1 which ran from Trenton to the westerly boundary of Elizabeth. This extension was, therefore, to run from Rahway Avenue, at the westerly boundary of Elizabeth, to the Jersey entrance to the Holland Tunnels in Jersey City.

On Fig. 1 is shown the original "Advisory Board" route and the location finally adopted for Route 25. It is worth while noting how physical and other conditions affected the location as well as how economic factors were applied to certain details.

Governing Points.—Doubtless if only through traffic traversing the whole length of the route were to be considered, in the quantity of 15 000 000 to 20 000 000 vehicles per annum, and if only the economic factors governing the cost of moving it were taken into account, economic justification could be shown for a practically straight, almost level, line from the Holland Tunnels to at least the easterly edge of Elizabeth, passing through Jersey City in a deep-level tunnel.

This latter, however, was considered entirely impractical, inasmuch as this highway was required to serve the requirements of Jersey City, making connections with its street system. Along the westerly side of the top of the hill (Fig. 1) there already runs a main north-and-south artery, the Hudson County Boulevard, with which connection also was desired. It was further necessary to provide for joining the new north and south State Highway (Route 1, 1927) which it was expected (in 1924) would come in from the north connecting the 178th Street Bridge across the Hudson with the proposed Bayonne-Staten Island Bridge.

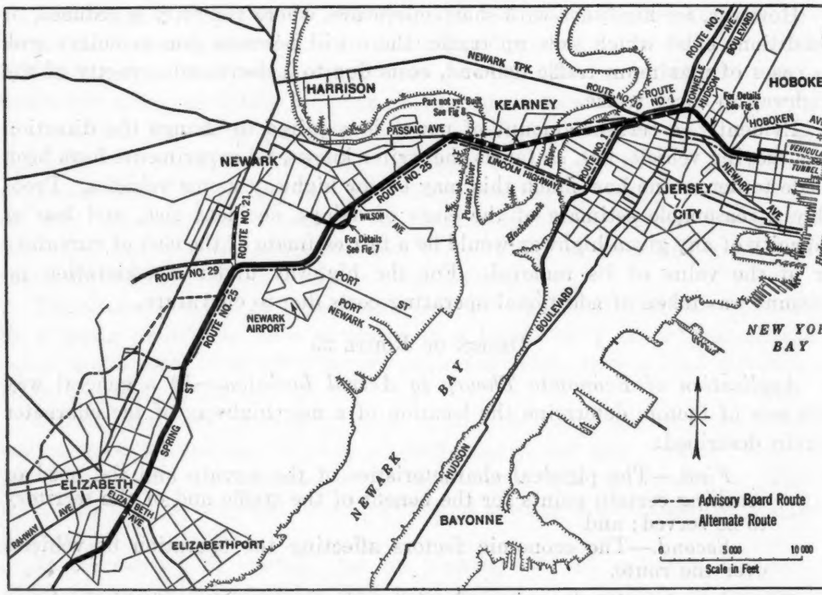


FIG. 1.—MAP OF JERSEY CITY, NEWARK, AND ELIZABETH, N. J., AND CONTIGUOUS TERRITORY, SHOWING LOCATION OF VARIOUS ROUTES.

These considerations determined the fact that the line would have to go over, instead of through, the hill, and an inspection of the profile, Fig. 2, will show that, in order to avoid crossings at grade with the Jersey City streets, crossings under the streets would probably be preferable to crossings over them. The maximum gradient of 3.5% on both sides of the hill just permitted the line to reach the elevation necessary to go under the streets. It also permitted a reasonable solution of the ramp connections to the main cross-streets (Fig. 3).

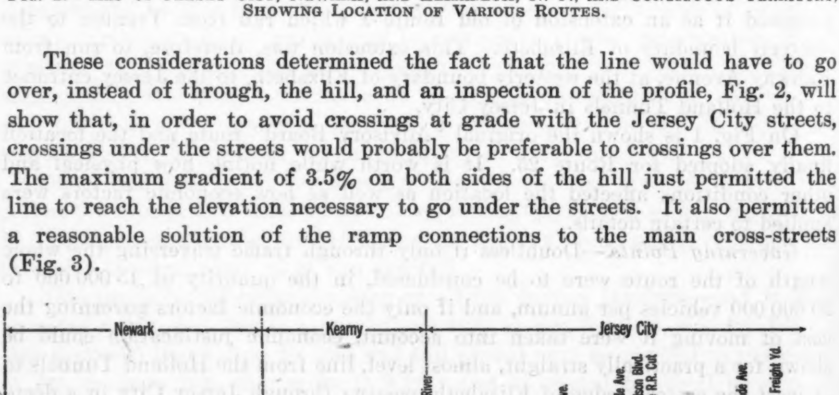


FIG. 2.—GENERAL PROFILE, ROUTE 25, FROM THE HUDSON RIVER TO ELIZABETH, N. J.

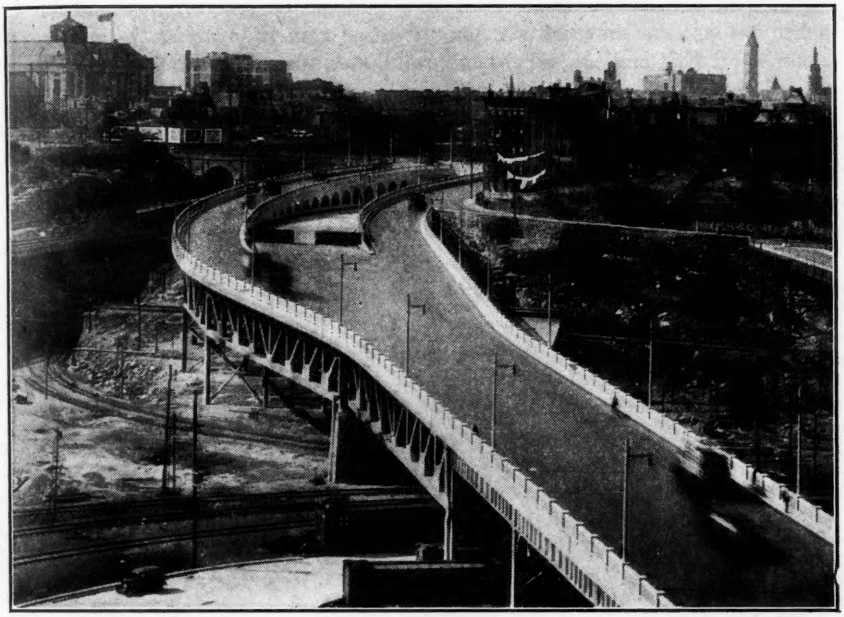


Fig. 3.—General View of Viaduct, Hudson County Boulevard, Looking Northwest in Jersey City, N. J.

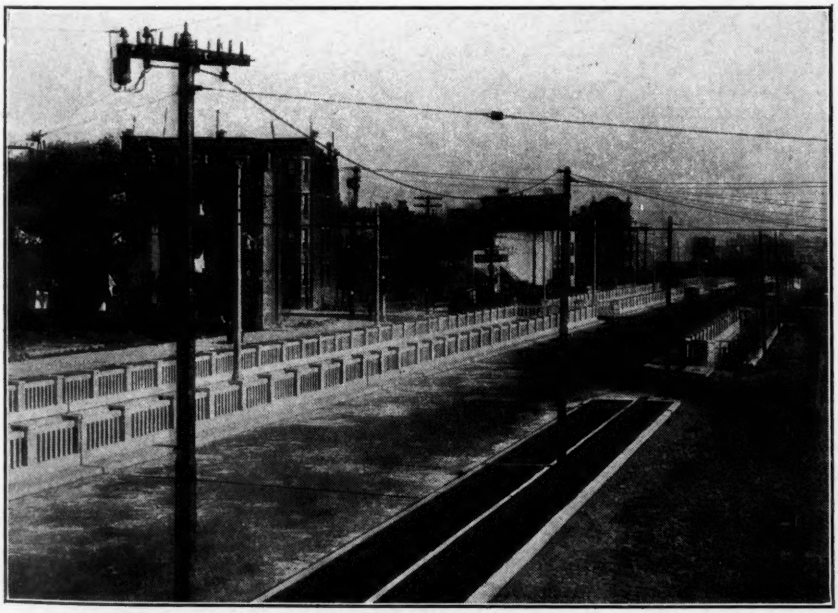


Fig. 4.—General View of Covered Cut, Hudson County Boulevard, Jersey City, N. J., Looking East.

If, however, further justification were required from the point of view of economics of operation, it may be noted that, for an overhead line, the difference in elevation would have been approximately 40 ft., and this alone would have justified an additional expenditure of $91 800 × 40 = $3 672 000, for the lower-level line. No calculations were made in this instance to show the difference in construction cost between an overhead and an underground line, because it was obvious that the lower line permitted easier solutions of the problems of connection by ramps to the important north-and-south highways. Also, it was fairly evident that an overhead line could not have been built for $3 672 000 less than the underground line which actually was built. Furthermore, an overhead line would have been objectionable from an æsthetic point of view.

Careful consideration was given to the problem of ventilation of the lower-level line. It was thought that, with openings at the southerly side for at least one-half the length and the overhead ventilation space on the southerly side, artificial ventilation would not be necessary—and experience has shown this assumption to have been been correct. The ramp connection at the top of the viaduct and on the easterly side of the hill is shown in Fig. 3. Other interesting details of this underground line are clear from Figs. 4 and 5.

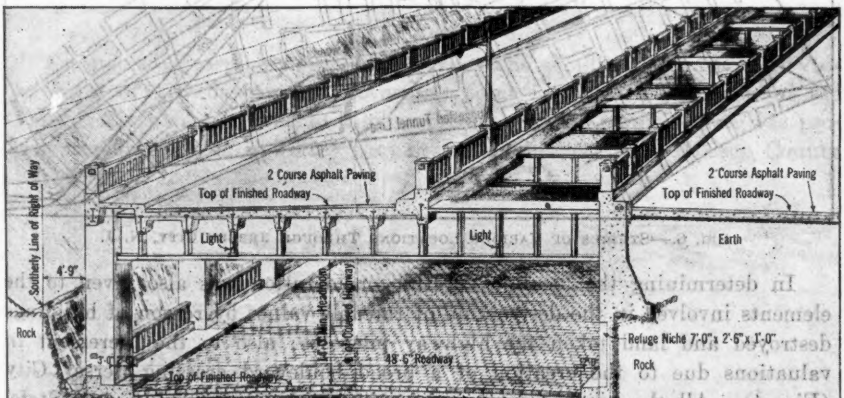


FIG. 5.—TYPICAL SECTION OF DEPRESSED HIGHWAY THROUGH JERSEY CITY, N. J.

ALIGNMENT AT EASTERN END

Location Through Jersey City.—Of the various lines for the route through Jersey City (Fig. 6), the "Advisory Board" line was objected to by the authorities of Jersey City, because it was proposed as an open cut and it was felt that the two already existing open cuts, of the Erie and Pennsylvania Railroads, had created undesirable barriers across the city. Further study of the character of the line also indicated that the selection of Broadway (on the west side of the city) for the route was not entirely desirable.

The Erie Railroad Company objected to the use of its property on the south side of 12th Street because of certain plans it had for warehouse development there. This brought up questions of right of eminent domain as between the Railroad Company and the State, which, if carried through the Courts, as they might have been, would have indefinitely delayed the work; so that, even had the question been resolved in favor of the State, the delay made some reasonable compromise desirable.

Studies were made, as shown by Fig. 6. The Railroad Company desired to maintain sufficient right of way, so that its cut might be enlarged to provide for eight tracks; but, finally, after prolonged negotiations with the City and the Railroad Company, the filing of injunctions, etc., an agreement was reached resulting in the location shown on Fig. 1. This provided that the highway be located alongside of and to the north of the Erie cut, that it be covered, and that it have a roadway on top (Figs. 4 and 5).

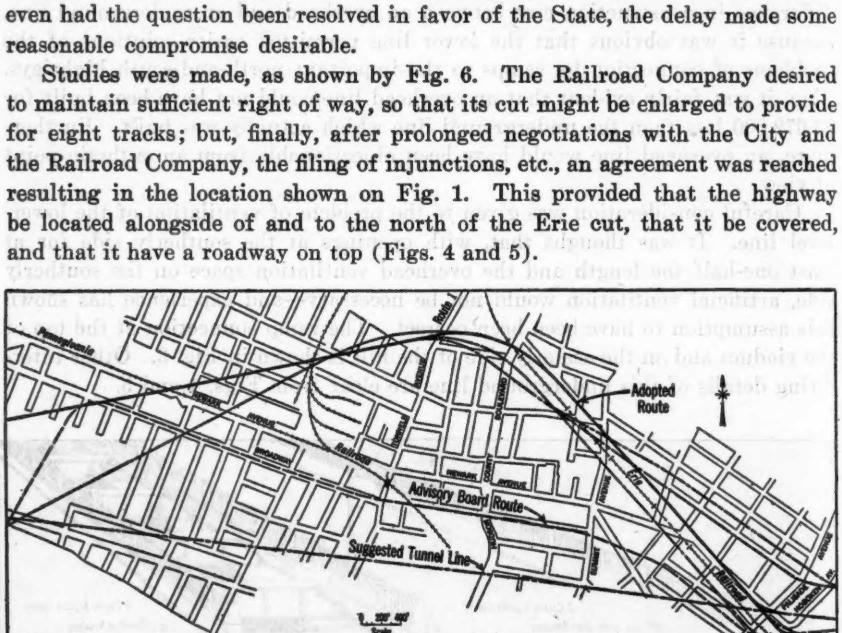


FIG. 6.—STUDIES OF VARIOUS LOCATIONS THROUGH JERSEY CITY, N. J.

In determining the location careful consideration was also given to the elements involved in the destruction of rateable values by reason of buildings destroyed and land taken for highway purposes; also to the increment in valuations due to the creation of a broad boulevard crossing Jersey City (Fig. 4). All the governmental entities involved, City, County, and State, are vitally affected by these elements and they exercise a considerable influence in the determination of locations.

The decision finally arrived at required a deviation from the general straight direction of the route, this being justified by several reasons: First, it was desirable to meet the demand of Jersey City that no additional barriers be built across the city, such as the open cut first proposed by the "Advisory Board". The controlling argument, however, was that proper connections between Route 25 and the Hudson County Boulevard and Tonnele Avenue could be made much more satisfactorily at or near the point where the line, as finally located, crossed them than elsewhere. A further reason was the possibility of obtaining a location through that part of Jersey City west of the Boulevard on which the right of way would not be unduly expensive and on

which grade crossings could be entirely avoided. No other location was as favorable in these respects.

Secondary Governing Point in Jersey City.—All these considerations led to the establishment of a secondary governing point in Jersey City in the vicinity of the crossing of the Hudson County Boulevard and the Erie Railroad. It seemed possible then to draw a straight line for the location from this point to a point on the easterly boundary of Elizabeth.

Reconnaissances through Elizabeth had indicated this second point at approximately the location of the line as now built through Spring Street. This westerly objective, or governing point, however, was modified almost at once by the need of reaching the locality where Routes 21 and 29 now join this highway (Fig. 1). In some degree this was influenced also by the fact that the City of Newark had acquired and partly graded a right of way from near the airport to the Elizabeth line, which could be utilized for the highway.

It developed further that the Lehigh Valley Railroad Company owned a right of way between its Oak Island Yard and the Newark Plank Road which it was willing to dispose of. Inasmuch as this right of way followed along the edge of the upland (that is, was on the upland), whereas the straight line was wholly on meadow land (marsh) this right of way as part of the route was given careful consideration, and finally adopted.

Other Problems

Secondary Governing Point in Newark.—The adoption of the Lehigh Valley right of way developed a governing point in Newark just west of the Passaic River on the Lincoln Highway from which a straight line was projected to the location in Jersey City near the crossing of the Hudson County Boulevard.

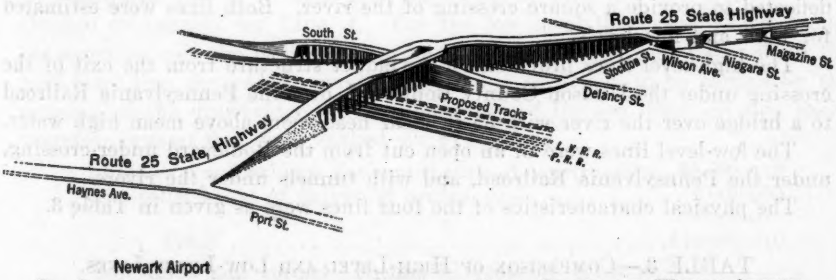


Fig. 7.—Streets, Connections, and Ramps at South Street and Wilson Avenue, Newark, N. J.

A connection to the Newark streets system was made between Wilson Avenue and South Street (Fig. 7). In working this out the State took over an existing contract between the City of Newark and the Lehigh Valley and Pennsylvania Railroad Companies which involved the abandonment of Port Street, thus permitting the development of the Oak Island Yards of these railways which it crossed at grade, the State Highway at this point providing the principal means of access between the City of Newark, the Newark Airport, and Port Newark. This is an important example of effective co-operation

between the local political and commercial interests, the railways, and the State, and between local and through traffic.

Factors Other Than Economic.—In considering all the physical, political, and other factors described, it may be felt that they entirely overshadowed those affecting the economics of operation. These latter, however, were brought continuously to bear in estimating the values of one or another proposed line. The writer is firmly convinced that these economic aspects are important elements which no conscientious engineer can afford to ignore, but the other elements have been set forth to emphasize that both aspects must be given due regard and their relative values properly appreciated.

Application of Economic Factors.—As an indication of the use to which these economic factors were put, there are cited here some comparisons made of two lines which were projected between the Hudson County Boulevard and the west side of the Hackensack River.* These Lines *A* and *B* are shown on Fig. 8 and the various profiles studied on Fig. 9.

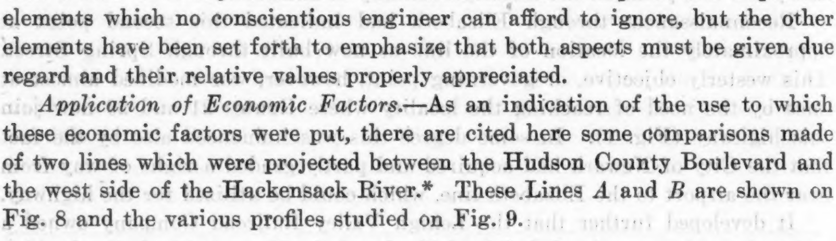


FIG. 8.—PART OF ROUTE 25, SHOWING ALTERNATIVE LOCATIONS ACROSS HACKENSACK RIVER.

Line *A* was a straight line (practically on the location afterward adopted), which, however, required a long skew crossing of the river. Line *B* was deflected to provide a square crossing of the river. Both lines were estimated for high and low levels.

The high-level lines provided for a viaduct structure from the exit of the crossing under the Hudson County Boulevard, over the Pennsylvania Railroad to a bridge over the river with 35-ft. clear head-room above mean high water.

The low-level lines were in an open cut from the Boulevard under-crossing, under the Pennsylvania Railroad, and with tunnels under the rivers.

The physical characteristics of the four lines were as given in Table 3.

TABLE 3.—COMPARISON OF HIGH-LEVEL AND LOW-LEVEL LINES.

	HIGH-LEVEL.		LOW-LEVEL.	
	Line *A*.	Line *B*.	Line *A*.	Line *B*.
Length, in feet	12 253	13 003	12 253	13 003
Rise, going west, in feet	......	......	86.2	83.9
Fall, going west, in feet	35.70	35.70	160.4	158.1
Curvature, in degrees	60	168	60	168

* These estimates were made in 1924 and 1925 and applied to conditions assumed at that time. Later, in 1929, conditions, especially in regard to the War Department requirements for a bridge crossing on the skew line, made it necessary to revise the estimated costs of construction.

The estimated costs of construction were:

High-Level:

Line *A*	$8 898 410
Line *B*	8 085 780

Low-Level:

Line *A*	$11 694 223
Line *B*	11 099 869

The difference in the lengths of the two lines is about 750 ft., and it will be recalled that the justifiable expenditure to save 1 000 ft. in distance on this line, and for its expected traffic, was estimated at $6 670 000. It is clearly evident, therefore, that for either level, Line *A* is preferable to Line *B*.

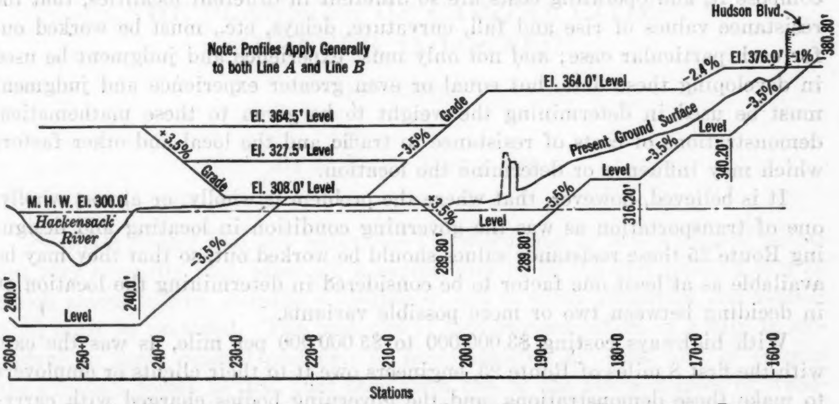


FIG. 9.—PROFILES SHOWING STUDIES FOR LINES CROSSING HACKENSACK RIVER.

There was then to be made the comparison between the high and low levels (bridge or tunnel) for Line *A*. For the low level there was added to the construction cost the capitalized value of the added rise and fall, and to the high-level line the added operating costs due to delays to be caused by the opening of the drawbridge. The comparison of the two lines on this basis, therefore, was at that time worked out as follows:

High level and bridge: Estimated costs of construction.	$8 898 410
Capitalized cost of delays at bridge................	8 000 000
Total	$16 898 410
Low level and tunnel: Estimated costs of construction	$11 694 223
Capitalized cost of rise and fall....................	9 780 000
Total	$21 474 223

On the basis of these assumptions, which were made in 1924-25, in connection with the earliest investigations of this problem, the high-level line then seemed to be preferable.

Since writing this paper, the profile of that part of the route across the Valley of the Hackensack and Passaic Rivers between Jersey City and Newark has been changed so as to provide 135 ft. clear head-room above mean high water at the river crossings. This change was brought about as the result of conferences to settle the long controversy as to the relative merits of

tunnels for these crossings as compared with drawbridges with 35 ft. clear head-room. The writer is entirely in accord with this final decision.

Modern Highway Demands

It has been impossible within the limits of a paper of this kind to fully demonstrate all the mathematical processes by which the data adopted for the studies for Route 25 were developed; but it is thought that the brief outline of their application to one section of the route will serve as a sufficient indication to experienced engineers, and no others should attempt to apply or develop them.

Highway traffic is so varied in the number and kind of vehicles which compose it, and operating costs are so different in different localities, that the resistance values of rise and fall, curvature, delays, etc., must be worked out for each particular case; and not only must experience and judgment be used in developing these data, but equal or even greater experience and judgment must be used in determining the weight to be given to these mathematical demonstrations of costs of resistance to traffic and the local and other factors which may influence or determine the location.

It is believed, however, that where the problem is wholly, or almost wholly, one of transportation as was the governing condition in locating and designing Route 25 these resistance values should be worked out, so that they may be available as at least one factor to be considered in determining the location or in deciding between two or more possible variants.

With highways costing \$3 000 000 to \$5 000 000 per mile, as was the case with the first 8 miles of Route 25, engineers owe it to their clients or employers to make these demonstrations, and the governing bodies charged with carrying out works of this nature owe it to the taxpayers not only to see that, for instance, the concrete is properly proportioned, the steel properly designed and erected, and the pavements adequate and properly laid; but before all this, to ensure that the general design of the route—that is, its location, both in plan and profile—is such that there is a true economic proportion between the costs of its construction and the costs of operation of the vehicles which have to use it.

Acknowledgments

The writer is indebted to the late A. M. Wellington, M. Am. Soc. C. E., and his "Economic Theory of Railway Location" for the basis of the theory herein applied to the location of Route 25.

The work was carried out under the general direction of the New Jersey State Highway Commission; Gen. Hugh L. Scott, U. S. A. (*Retired*), *Chairman;* and William G. Sloan, M. Am. Soc. C. E., State Highway Engineer. Great credit is to be given to all of these for their far-sightedness in the original conception of the general features of this highway and their initiative, co-operation, and help in the solution of the problems involved, both these herein referred to and innumerable others.

The writer also gratefully acknowledges the assistance of S. Johannesson, M. Am. Soc. C. E., who not only worked out the mathematical solutions of these problems, but who also was in entire charge, under the direction of the writer, of the design of the structure.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

THE PLANNING OF CAPITAL CITIES: DENVER, COLORADO

Discussion*

By S. R. De Boer,† Esq.

S. R. De Boer,‡ Esq. (by letter).§—The discussion by Mr. Knowles‖ points out very correctly that, after all, a capital city is, in the first place, a city in which places to live, to make a livelihood, and to enjoy education and recreation, must be provided. The presence of Government offices in a city is secondary to these general problems of municipalities at large, and in essence the question comes to this, that they form one of the many sources of livelihood that a city must have. The importance of the Government center decides whether or not its influence on the life of the city is an important one. There are in the United States a great number of small cities—State capitals—to which the State offices form one of the major single items. In the city's business life this must be fully acknowledged and to a certain degree the city should be built toward it. The average citizen of a Commonwealth may be ever so much opposed to taxation, but to have a capital city of which he must be ashamed is beyond his conception. In other words, it must perforce become a "dress-up" city that equals or surpasses others in the State in attractiveness and compares well with other capital cities.

Whether a capital city should not encourage industry and commerce must depend on the relative importance the Government offices hold in its business life. Nearly every capital city will find it to advantage to have some industry and commerce, and to such large cities as Boston, Mass., these overshadow the Government business to such an extent that the latter becomes of minor importance. Even in a small capital city, where the State offices are extremely important, industry can be safely encouraged if it is located in a

* Discussion of the paper by S. R. De Boer, Esq., continued from March, 1930, *Proceedings*.

† Author's closure.

‡ City Planning Consultant, City of Denver, Denver, Colo.

§ Received by the Secretary, April 28, 1930.

‖ *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2229.

carefully selected district. Any city is much better equipped by having many sources of livelihood rather than just one, but if the Government offices play the important rôle of being the main resource, the city authorities should fully recognize the fact and build to it.

Denver is to a great extent a governmental city and the administrative offices play an important part in its business life. It is not only the Capital of the State of Colorado, but its citizens consider it more or less a Western Capital of the Nation. It has a very large number of Federal offices and a tremendous territory is administered to from this center. In Denver's business life the Government offices have become of unusual importance through the fact that climate and mountain scenery have made it a National recreation center as well, in a sense similar to Washington, D. C. Tourists form a good adjunct to the business life of Colorado's Capital City.

Mr. Knowles' reference to the industrial survey is timely. Too many cities have concentrated on getting industries at the expense of neglecting the building of a desirable city. The aim must be, in the first place, contented, healthy and educated citizens. If for this purpose more industry and business are necessary, well and good, but to encourage the latter for the mere sake of greater population and higher real estate values is as absurd as it is frequent. In generations to come many cities which have overdone industrial efforts may have to demolish what has been built now, in order to create better living conditions. The writer agrees fully with Mr. Knowles that the city engineer was the first city planner. Many cities owe their conditions, good or bad, to the city engineer. He has considered every improvement in its value to the city as a whole, and he has done this work with a loyalty and earnestness that characterizes the profession. His main handicap has been the lack of a study treating the whole city as one entity, rather than many separate parts and in such a way that fundamental lines of development are laid down for a long period to come. The few engineers who did follow these lines must have realized the difficulty of getting elected councils to follow their imagination. The present age with its highly specialized work is more inclined to relieve the city engineer from the burden of city planning and leave this to specialists, and thus allow the engineer more time for his already complicated work.

In many respects Denver has problems that are the very opposite of those of other communities. Many cities on lake or ocean fronts are battling the problem of drainage. Denver is a mile above sea level and is the heart of the greatest semi-arid region in the country. Farm lands must be irrigated in order to yield heavy crops. The city is surrounded by large areas of these irrigated farms and they stabilize its growth and real estate values. Great care is exercised by the water authorities to see that in the growth of the city this farm water is not used for domestic supply. The domestic supply returns to the river bed several miles below the city limits; a use of the water from farms above would mean the shifting of irrigated land with a possible shifting in the growth of the city. Arrangements are being made to bring water through the Moffat Tunnel from the western slope of the Rocky Mountains for use in Denver. This water will not be needed for many years, however, and

the additional storage now being planned will provide water from the eastern slope sufficient for probably 25 to 50 years' growth.

In Mr. Lyman's discussion* the very interesting numbering system of Salt Lake City, Utah, is described. This system is undoubtedly of great practical value. Many cities have followed the plan of naming streets after their leading men. There is a more human factor in this practice than in mere numbering. It has often seemed to the writer that the names of streets should indicate to what they lead, such as Capitol Avenue, University Avenue, etc. This would not be practical for large numbers of streets, but would well be possible for the more important avenues.

The plan of Salt Lake City, with its blocks 660 ft. square and its streets 132 ft. wide, presents an unusual problem in city planning. The wide streets are of great value in present-day traffic. Just how far the interior of the large blocks will prove to be practical the future must tell.

In his discussion,† Mr. Herrold confirms the thought that the writer tried to express, namely, that the people in a capital city must clearly understand its function; they must realize its position in the eyes of the citizens of the State, and they must strive to build to attain that position. A State civic center with broad parkways leading to it embodies considerable of this thought. Denver has undertaken this in its civic center, which is a continuation of the Capitol grounds. The two have been developed in such a way that a harmony exists, which makes a general State center out of them.

The suggestion‡ embodied in Mr. Herrold's discussion of broad parkways leading to the capitol buildings and fronting them with buildings, such as lodges, churches, etc., is a very attractive one. The time will come, perhaps sooner than can now be predicted, that such semi-public buildings can be forced to a location of this kind by zoning provisions. Some zoning men frown at this statement, but the step to a definite regulation of this character, with many more of its kind, is much shorter than the original step to zoning.

The City of Cheyenne, Wyo., is working more or less in this direction. It is constructing all its lodge buildings and some of its churches on Capitol Avenue; it has plans for a State center. Some of it is a conscious effort and some perhaps subconscious, but the results are very promising.

In closing this discussion on capital cities it may be of value to recapitulate the governing ideas brought out:

First.—A capital city must recognize its position in the State and Nation and try to express the pride of the citizens of its political entity.

Second.—It must carry out this idea with, and not in spite of, its following the usual building program of any city.

Third.—It can encourage industries and commerce, but must be careful in their location on the plan of the city.

Fourth.—It must lead its State or Nation, not only in beauty of civic buildings, but also in beauty of residential and business sections.

Fifth.—It must lead where others can be contented to follow.

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* *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2409.

† *Loc. cit.*, March, 1930, Papers and Discussions, p. 623.

‡ *Loc. cit.*, p. 625.

It seems fitting at this time to recall the memory of the late Charles Backus Ball, M. Am. Soc. C. E., to whose efforts so much of the success of the City Planning Division was due. Each program of the Division, including the one at which this paper was presented, was largely the result of his initiative and foresight and was made concrete by his untiring efforts. Mr. Ball died on October 18, 1928, fifteen months after this paper was first presented. Despite his advanced age, this wonderful engineer outshone many young engineers in enthusiasm and activity, and his great store of information was invaluable to all.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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ADJUSTMENT OF TRANSIT AND STADIA TRAVERSES

Discussion*

By Howard S. Rappleye,† Assoc. M. Am. Soc. C. E.

Howard S. Rappleye,‡ Assoc. M. Am. Soc. C. E. (by letter).§—The writer is indebted to Professor Underwood|| for a decided improvement in the method of adjusting transit and stadia traverses, since his proposed method of weighting the various courses will undoubtedly result in better corrections. This method will also tend to remove the objection advanced in Professor Stewart's discussion.¶

The discussions by Mr. Peters** and Mr. Parry†† present valuable information on the subject and serve to show that the treatment of the problem was checked, so to speak, before it was solved. It is superfluous, perhaps, to state that the writer had no knowledge of the work of the Canadian engineers in connection with this problem, until their discussions appeared.

The matter has been presented to the attention of the engineers of the country, much improved as a result of its discussion by others. It will be interesting to sit back and see what practical use is made of the method.

* Discussion of the paper by Howard S. Rappleye, Assoc. M. Am. Soc. C. E., continued from April, 1930, *Proceedings*.

† Author's closure.

‡ Assoc. Mathematician, Div. of Geodesy, U. S. Coast and Geodetic Survey, Washington, D. C.

§ Received by the Secretary, May 20, 1930.

|| *Proceedings*, Am. Soc. C. E., April, 1930, Papers and Discussions, p. 847.

¶ *Loc. cit.*, January, 1930, Papers and Discussions, p. 175.

** *Loc. cit.*, April, 1930, Papers and Discussions, p. 849.

†† *Loc. cit.*, p. 851.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

FACTORS GOVERNING THE LOCATION OF AIRPORTS

Discussion*

By Messrs. William J. Fox, Perry A. Fellows, W. W. Crosby, E. K. Smith, and Donald M. Baker.†

William J. Fox,‡ Assoc. M. Am. Soc. C. E. (by letter).§—Air transportation has so fixed itself into the modern method of traveling that it is now an established economic necessity.

The time element factor which differentiates the automobile from the airplane is so great that, economically, the business man can no longer afford to travel long distances by automobile. The time element in modern business is of such importance that this one factor alone determines the success or failure of the business structure. To no small degree has this same element entered into present-day social and recreational existence.

The fact that transportation is necessary at all is, basically, a handicap to any business. As an example, two businesses of identical character, one requiring transportation facilities for its operation and the other so located as to not require such facilities, other things being equal, it will be found that the transportation factor determines, in terms of time and cost, the financial differential of the two enterprises. Therefore, the form of transportation which minimizes most successfully the time element, with a corresponding reduction of cost, and which more closely approximates the condition in which no transportation is necessary, is of the greatest economic value. Aviation or air transportation is and no doubt always will be the most effective in meeting the foregoing requirements.

***Safety in Aviation.*—Taken in relation to certain facts, the present mortality arising from air travel is surprisingly small. The vital statistics of the United States, England, and Wales indicate that the number of deaths in aviation per 1 000 000 of population is quite uniform.**

* Discussion of the paper by Donald M. Baker, M. Am. Soc. C. E., continued from April, 1930, *Proceedings*.

† Author's closure.

‡ Chf. Engr., Los Angeles County Regional Planning Comm., Altadena, Calif.

§ Received by the Secretary, March 7, 1930.

The airplane, in the main, has gone through throes of the experimental stage in so far as structural characteristics are concerned. Airplane motors have been perfected to the same degree as the most finely finished watch. Accidents due to structural defects are an extreme rarity. Only 5% of the total number is due to this cause. The fact seems to be that passengers, on regular air lines owning perfect machines that are under the control of regularly licensed pilots, enjoy a good degree of safety.

The airplane undoubtedly has achieved the highest place as a medium for transportation, especially for long distances. The financial advantage of utilizing this form of transportation for both passengers and light cargoes is strongly manifested by the air transportation companies that have been formed during the past few years and by the tremendous amount of capital being invested in the aviation industry.

TABLE 2.—Comparative Mortality in Air Travel.

	Year.	Mileage.	Fatalities.	Average per 1 000 000 population.
United States	1926	....	160	1.5
United States	1927	....	214	1.6
United States (first six months)	1928	....	161	1.6
British Air Lines	1925–1927	2 471 000	None	
British Air Lines	1922–1924	2 596 000	15	1.4
Luft Hansa, European	1925–1927	12 605 000	22	1.7
United States	1921–1926	....	903	1.5
England and Wales	1921–1926	....	265	1.1

The success of the airplane as a safe and efficient means of transportation is largely dependent upon the selection and design of the airport on which it must land and take off. Airport location looms as one of the chief factors involving safety as well as the future development of aviation. The present airports in American cities in general, even those of the largest size, are being adjudged as inadequate and inefficient.

Selection of Modern Airports.—To gain public confidence in this form of transportation, airport sites must be selected on the basis of engineering judgment coupled with the advice of flying experts. They should be laid out to the best advantage, with the aid of engineering skill.

If aviation is to grow as it has a right to grow, it must enlist the services of the flyer himself, the civil engineer, the architect, and the city planner, as it has enlisted the services of the scientists, the navigator, and the motor expert.

One or two serious accidents to passenger planes, due to the unwise selection or faulty design of an airport, has a disastrous effect upon the aviation industry as a whole. The right to fly air mail over a community might even be put in jeopardy. In view of the position that aviation now holds in the economic structure of the United States, it is essential that public confidence should be augmented rather than impaired.

The selection of a field is more than a real estate transaction. Competent men who are engaged in the actual flying of aircraft should be consulted in

order that the particular field under study be given the official endorsement of those men who will be called upon to fly planes to and from the airport when selected.

Technical Advice.—Mr. Baker emphasizes* the importance of certain factors in determining the proper location of airports. It is imperative that factors be used which will tend to give an impartial rating by a purely mathematical or mechanical process. These factors should be a minimum in number, but at the same time representative of the vital characteristics of a safe and efficient airport. Quite as important as the factors themselves is the procedure followed in leading up to their application, for example: (*a*) Survey of the area to be served; (*b*) technical advice; (*c*) future predictions of population and airplane growths; (*d*) economic analyses; and (*e*) application of factors by qualified experts.

The lack of technical advice is not only expensive in the initial selection of the field, but poor judgment in the selection as to proper location usually entails great cost in order to modify conditions to make the airport safe and adequate. The cost of adjusting topographical conditions, the elimination of hazards and the installation of drainage systems on a poorly selected field runs up the cost to such an extent that it might have been more profitable to have selected a site, the initial cost of which was far greater. Likewise, the adequate laying out of the development plan for an airport is, in many respects, similar to planning a small city.

The matter of controlling the height of buildings constitutes a strong factor. Height limit regulations in the vicinity of a field give the operators assurance that they will have the best possible use of their field; they also contribute added safety to those using the air as a means of transportation.

Selection with Respect to Growth Trends.—Airport selection and construction require a careful study of land areas, growing and shifting population, industrial and residential development.

The selection of an airport site is properly a part of city planning, and as such should be in conformity with the future development program. As part of its planning activities, every city should prepare an airport development program that will provide for future needs on the basis of real economy.

The airport is the gateway to the modern city. If properly selected, a satisfactory financial return can be shown for the investment involved. To make the investment economically sound, however, the following principles must be rigidly adhered to:

1.—The site should be selected by qualified experts.
2.—The land should be owned in fee or on long-term lease.
3.—The airport should be accessible to those served by a maximum of 20 min. surface transportation time.
4.—The field should be of the highest rating commensurate with use.
5.—Proper legislative control should be enforced over surrounding development.
6.—The field should be properly related to the community served and other fields.

Los Angeles County Method of Locating Airports.—In the latter part of 1927 Los Angeles County made the study of airports, landing fields, and

* *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2216.

various ground requirements a part of the activities of the Regional Planning Commission. This Commission is a part of the County Government and is charged with the duty of planning the wholesome development of the County as one great commonwealth. The area involved was slightly less than 5 000 sq. miles, including forty-four incorporated cities and thirty-six unincorporated towns. It was obvious that thorough planning had to embrace the requirements for this new form of transportation. Otherwise, any shortcomings would be discovered too late to adjust the development economically to the situation.

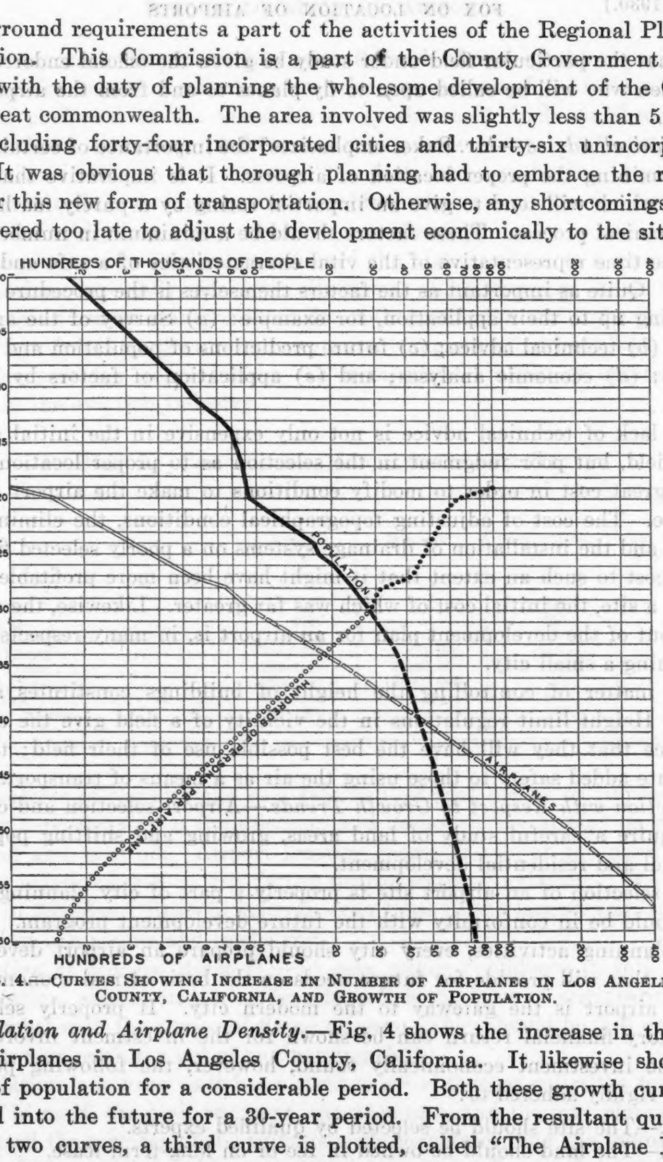

FIG. 4.—CURVES SHOWING INCREASE IN NUMBER OF AIRPLANES IN LOS ANGELES COUNTY, CALIFORNIA, AND GROWTH OF POPULATION.

Population and Airplane Density.—Fig. 4 shows the increase in the number of airplanes in Los Angeles County, California. It likewise shows the growth of population for a considerable period. Both these growth curves are projected into the future for a 30-year period. From the resultant quantities of these two curves, a third curve is plotted, called "The Airplane Density Curve", which represents the number of persons per airplane as of the past and as anticipated in the future.

The present status in this County (1930) is one airplane to every 2 500 persons. It is estimated that by 1980 this ratio will be 500 persons per airplane, and in the year 2030, 100 persons per airplane.

With the population curve as the basis and the 20 min. of surface transportation as a maximum figure, some basic information is provided upon which to calculate the size and frequency of the airports that will be needed to satisfy future needs.

Due to the very rapid growth experienced, it was evident that the improper location of one road might forever destroy a dominant site for a future airport or landing field. The time to locate airports is before land is subdivided or cut up by streets or highways. Therefore, a study of the requirements that would be needed for the future of aviation was made in conjunction with the plans and studies for highways, population densities and trends, industrial development, transportation facilities, zoning, and land subdivision. The results have been co-ordinated into a master plan. As divisions of the Master Plan are completed they are submitted for official and public approval. The entire procedure involves six steps, namely, (1) to take an inventory of existing conditions; (2) to form an aeronautical advisory committee; (3) to make a comprehensive study of available sites; (4) to estimate the cost of equipment and improvements for airport operation; (5) to analyze the financing and probable earning ability; and (6) to design the airport.

Step No. 1.—Inventory of Existing Conditions.—A survey and inventory were made of all existing airports, landing fields, and aviation facilities within the County. With respect to each existing airport information was obtained as to the following factors:

(1) Location.
(2) Description.
(3) Runways and landing areas.
(4) Obstructions and hazards.
(5) Equipment.
(6) Improvements.
(7) Conditions of ownership.
(8) Class.
(9) Area.
(10) Elevation.
(11) Drainage conditions.
(12) Soil conditions.
(13) Meteorological data.
(14) Transportation facilities.
(15) The name of the operator.

The survey revealed that there were fifty-two fields in operation. They were of all sizes, shapes, and conditions of ownership. The relative sizes were found to be as follows:

Size, in acres.	No.
20 to 30	15
30 to 50	6
50 to 100	7
100 to 160	7
250	1
380	1
640	1
Not determined	14
Total	52

The acreage was variously owned, as shown in Table 3.

TABLE 3.—COMPARISON SHOWING NATURE OF OWNERSHIP.

Nature of ownership.	Number of acres.	Number of fields.
Public	784	5
Private	570	8
Leased	227	39
Not classified	2 050	
Total	3 631	52

Many of the fields were operating on leases of from thirty days to ten years, as shown in the following list:

Length of lease.	Number of airports.
30 days	20
60 days	2
90 days	3
1 year	2
2 years	1
3 years	4
5 years	4
10 years	3
Not leased	13
Total	52

This short-term lease condition greatly handicaps the developers and operators in securing loans or encouraging investments for expansion or permanent improvements.

Step No. 2.—Formation of Aeronautical Advisory Committee.—Although the Commission was equipped with a technical personnel of engineers, architects, landscape architects, and statisticians, it realized that it must be fortified with proper aeronautical advice to obtain a true cross-section of aviation problems and requirements. To meet this situation, an Aeronautical Advisory Committee composed of expert flyers from the Army and Navy, U. S. Department of Commerce, air transport companies, aerial police, and civilian flyers, was formed. The studies, designs, and accomplishments of the Commission were placed upon a basis as nearly scientific as possible.

Step No. 3.—Comprehensive Planning of Airports.—After all the existing fields had been surveyed, plotted on maps, and evaluated, the technical staff of the Commission divided the County, for the purpose of study, into the following natural physical divisions: (*a*) Antelope Valley; (*b*) San Gabriel Valley; (*c*) San Fernando Valley; (*d*) La Crescenta Valley; (*e*) the central area of Los Angeles; (*f*) Southeast Section; and (*g*) Pacific Coast Beach Section.

With the aid of maps showing the existing and proposed highway systems, other maps showing land that was subdivided, and with U. S. Geological Survey maps showing topography (scale 1 in. = 2 000 ft., with 5-ft. contours), the technical staff indicated every flat parcel of land that had any possibilities whatsoever as a potential airport.

Each physical division was first studied separately as a unit. The Aeronautical Advisory Committee assigned six qualified pilots and observers the job of flying over a given sector of each physical division, with instructions to select sites independently from the air and to make a full aviation survey and report on the possible sites selected by the Commission's staff. An engineering analysis was made of each site, and to this analysis was added the findings of the observers and pilots. Each site was then given a rating. The process of rating was purely mechanical and relative. The factors applied were as follows:

Surrounding topography	10
Physical condition of site*	20
Location with respect to other fields	10
Transportation facilities	10
Accessibility	15
Utilities	5
Local hazards	15
Drainage	5
General desirability	10
Total	100

These factors correspond very closely with those recommended by Mr. Baker. When evaluating each factor for a site under study, the conditions which result in a certain rating should be elaborated upon, and their relativity established. For example, if the drainage cost on five airport sites was \$3 000, \$5 000, \$25 000, \$4 000, and \$10 000, respectively, it is obvious that the one costing \$3 000 would be the cheapest, and would warrant the maximum rating of 5.00 points. The other ratings would then be 3.00, 0.60, 3.75, and 1.50, respectively.

The engineering analysis of a site followed a procedure which analyzed the physical characteristics, cost of grading, drainage, flood protection, and removal of hazards, its relation to present and future population, its status as part of the Master Plan, and a financial analysis of the earning ability of the field. This engineering analysis was separated into five divisions.

The financial analysis and estimated cost of putting Air Terminal No. 8 in condition† for airport operation, was as follows:

		Percentage of total.
Division 1.—Physical Analysis of Site:		
Total area of site, in acres	968	
Total acres that can be used for airport purposes if improvements in Division 2 are carried out	935.73	96.67
Acres subject to overflow of river and washes	32.27	3.33
Area (acres) unusable for airport purposes	32.27	3.33

Division 2.—Estimated Cost of Improving Site:		
Removing 877 acres of walnut trees, buildings, irrigation lines, etc.	\$40 000	
Grading (west of flood control levee) 100 000 cu. yd. of cut and fill	50 000	
29 000 lin. ft. of fence	29 000	
Drainage	5 000	\$124 000
Division 3.—Outstanding Bonded Indebtedness:		
El Monte School	\$16 629	
Mountain Avenue School	342	
Flood control	5 351	
County improvement	19 722	\$42 044
Total		\$166 044

* Meteorological study was over the area as a whole.

† Does not include cost of land.

Brought forward.......................... $166 044

Division 4.—Estimated Cost of Equipment and Improvements for Airport Operation:

Hangars with concrete aprons..........	$200 000	
Lighting (flood lights, runway lights, boundary lights, beacon light).........	50 000	
Administration building and depot.....	40 000	
Comfort stations................................	3 000	
Turfing, 2 151 364 sq. yd. on runways and landing area...........................	129 082	
Water supply development (pumps and storage)..	50 000	
Telephone and telegraph system......	15 000	
Meteorological equipment...............	5 000	
Sprinkling system for landing area and runways....................................	35 000	
Road work.......................................	10 000	
Miscellaneous equipment (trucks, roller, blades, scarifier, tractor unit, ambulance and first aid equipment, fire equipment, and small tools)........	30 000	$567 082
Total		$733 126

Division 5.—Financial Analysis and Earning Ability.—The study of finances and probable earning ability are arranged in Table 4.

TABLE 4.—FINANCIAL ANALYSIS AND EARNING ABILITY, AIR TERMINAL NO. 8.

	1930	1940	1950	1960	1970	1980
Population (San Gabriel Valley)..............	250 000	480 000	625 000	750 000	900 000	1 050 000
Number of airplanes (San Gabriel Valley)...	62	480	2 080	5 000	8 500	10 000
Number of airplanes at terminal............	31	240	1 040	2 500	4 750	5 000
Number of hangars required..................	2	16	69	166	316	234
Square feet of hangar space.................	40 000	320 000	1 380 000	3 320 000	6 320 000	6 680 000
Rental from hangar space......................	$ 1 200	$ 9 600	$ 82 800	$139 200	$259 200	$400 800
Revenue from manufacturing; accessories, etc..	2 000	8 000	20 000	40 000	80 000	160 000
Revenue from taxi schools, landings, transports, etc..	300	2 400	20 700	25 000	27 000	26 000
Revenue from utility service.................	500	3 000	5 000	7 000	9 000	11 000
Revenue from office service..................	500	5 000	12 000	20 000	30 000	40 000
Miscellaneous revenue.........................	100	500	2 000	5 000	8 000	10 000
Total revenue	$ 4 600	$28 500	$142 500	$236 200	$413 200	$647 800
Net profit from walnut crop...................	$24 672	$24 672	$ 24 672	....	....	....
Gross annual earnings...........................	$29 272	$42 172	$167 172	$236 200	$413 200	$647 800

Summary of Financial Analysis.—The foregoing items may be summarized as follows:

(A) Annual Earnings for 50-Year Period, 1930-1980:

Net annual profit on walnuts = $82.24 per acre (average) × 300 acres...........	$24 672	
Average annual earnings over 50-year period.		$275 460
Average annual cost of operation and maintenance	$30 000	
Annual interest on $733 126 (improvements) plus $2 500 000 (land) at 6%............	193 987	223 987
Average annual net profit (50-year period)............		$51 473

(B) Ultimate Annual Earning Capacity of Site No. 8:

Anticipated ultimate population, San Gabriel Valley (Section 2-E)	3 000 000
Anticipated airplanes in 1965, San Gabriel Valley	6 000
Anticipated airplanes when San Gabriel Valley reaches 3 000 000 population	30 000
174 acres of hangar space at 3 cents per ft. per year	$227 380
Depot and administration office space rental, 50 000 ft. at $1.00	50 000
100 acres available for shops, accessory houses, manufacturing plants, etc., at 10 cents per acre	435 600
Expected annual revenue from concessions, landing rentals (student pilots), lockers, transport passenger tax and miscellaneous	50 000
Ultimate annual earnings	$762 980

As a result of this study, Air Terminal No. 8 was given the following rating:

Surrounding topography	10
Physical condition of site	20
Location with respect to other fields	10
Accessibility	13
Transportation facilities	10
Utilities	3
Drainage	5
General desirability	10
Local hazards	12
Total	93

Analyses and studies similar to the foregoing were made for each site. Then the best were selected, by the process of elimination, to serve the County in a comprehensive way with airport facilities. The comprehensive plan was so designed and arranged that the existing airports which had received a satisfactory rating were properly recognized. The proposed airports were so arranged and situated as to serve the ultimate population and were so chosen that the development of the sites themselves could be accomplished if and when they were in demand. The official highway plan of the County was adjusted to conform to and preserve both the existing and the proposed airports.

The general procedure was to select, at a strategic location in each physical division of the County, a major air terminal, from which would operate transport planes, cargo planes, and all large aircraft.

Definition of Major Air Terminal.—A major air terminal is an airport of sufficient size to permit the safe and efficient operation of the largest transport planes that are now in service or likely to be placed in service. It must have a runway of at least 5 000 ft. in length and 400 ft. in width, or must be so equipped with apparatus or mechanism as to permit the taking off of planes in like number and with the same degree of safety as would the runway stipulated. It must also be able to provide a separate area that will permit landings in all directions. The dimensions of such landing area must be at

least 2 500 by 2 500 ft., or must be equipped with apparatus or mechanism to accomplish the same effect in the matter of safety and number of landings as would the area stipulated.

A major air terminal must have sufficient area to enable the erection of storage hangars for at least 1 000 airplanes; it must have adequate space for shops, manufacturing plants, and a passenger and freight depot; and it must be adequately served by both rail and highway transportation facilities. It must also be so located as to serve, adequately, a large population, either actual or potential. Furthermore, it must be able to warrant the highest class airport rating of the Aviation Division of the Department of Commerce.

In each physical division of the Los Angeles County survey there were selected satellite fields to serve each community. Private ships, police planes, Army and Navy ships, taxi planes, and small craft will operate from these fields.

Definition of Satellite Airport.—A satellite airport is a field so designed as to accommodate ships smaller and of less wing spread than transport planes. It must be of sufficient size to permit the construction of a runway of at least 1 800 ft. in length. It must also have hangar space for the storage of airplanes, such space to be commensurate with the population served.

The general purpose of a satellite field is to serve the owners of private planes, commercial taxi planes, light military planes, and police planes; also, to serve as emergency landing fields for larger planes in distress.

The satellite fields in Los Angeles County were so located as to be accessible by ground transportation not involving more than 10 min. of time from any point. In addition to serving the community with airport facilities for the present and future, in so far as private ships were concerned, these satellite airports give opportunity for aviation facilities for long-distance travel. This is accomplished by the operation of taxi planes from the satellite fields to the major air terminal, from which operate the large passenger ships. The distribution of air mail to local points is accomplished in a similar manner.

Fig. 5 illustrates the comprehensive airport development plan for Los Angeles County. As each physical division of the County is surveyed and reported on by the Commission's technical staff and its Aeronautical Advisory Committee, detailed plans are prepared for the design and development of each recommended site.

Step No. 6.—The Design of the Airport.—Attention is given to the design of the major air terminal for the reason that the area involved and the design of the field itself must be commensurate with the population served. The future population density gives a fair basis for the facilities required. At the outset it is recognized that the major air terminals will have the greatest concentration of aircraft operation. The area of the immediate physical division that each major air terminal serves ranges from 200 to 300 sq. miles and an ultimate population of from 1 000 000 to 3 000 000.

One of the chief factors in the safety of the airplane as a means of transportation is the proper and efficient design of the airport. In the design of all major air terminals one of the basic principles that has been followed in the

plan is to provide for the segregation of the landing area and the runways for taking off. Another is to have runways of sufficient length to permit landings on the field in the event of a "dead stick" or a "concked motor". It has been quite definitely established that these basic principles contribute to greater safety and efficiency. Such design will permit the simultaneous landing and taking off of a squadron of planes; it likewise permits the continuous landing or the continuous taking off of ships, on schedule or off schedule, with a large factor of safety and efficiency.

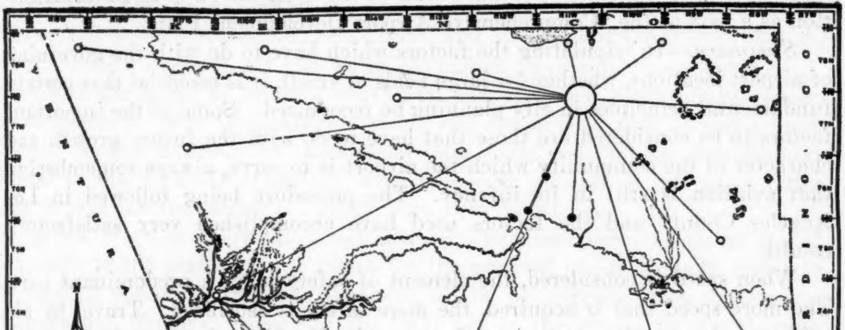

FIG. 5.—PRELIMINARY COMPREHENSIVE AIRPORT DEVELOPMENT FOR THE COUNTY OF LOS ANGELES, CALIF.

Application of the Comprehensive Plan.—It is generally recognized by those vitally interested in the future of aviation that it is essential that a systematic and comprehensive plan be followed. Those developing airports in Los Angeles County have the benefit of the impartial study and advice of the Regional Planning Commission and its Aeronautical Advisory Committee; and the selection of major air terminals or local fields, whether private or public, has closely followed the recommendations as set forth in the Master Plan.

The selection of a site in accord with the plan gives protection in the form of zoning; it provides for the proper construction and location of ground-transportation facilities, regulation of the height of structures in the vicinity of the field; and also guards against having the field declared a nuisance.

The financial set-up for the development of air terminals differs somewhat in each instance. If the purchase of any air terminal is to be made from public funds, it is the intention of the Regional Planning Commission of Los Angeles County that such purchase shall be made on the basis of its contribution as a part of the "Comprehensive Airport Development Plan".

Summary.—In stipulating the factors which have to do with the governing of airport locations, whether for large fields or small, it is essential that certain fundamental principles in city planning be recognized. Some of the important factors to be considered are those that have to do with the future growth and character of the community which the airport is to serve, always remembering that aviation is still in its infancy. The procedure being followed in Los Angeles County and the factors used have accomplished very satisfactory results.

When speed is considered, the element of safety plays a predominant part. The more speed that is acquired, the more safety is sacrificed. Travel by air will never be as safe as travel on the ground. Much can be done, however, to make air travel reasonably safe. The index of the safety factor now rests with the design, location, and adequacy of airport facilities. The majority of airplane accidents occur either when landing or taking off. If the same factor of safety can be incorporated in the design and location of airports as is accomplished in the design and construction of the plane itself and by the thoroughness of the pilots, aviation will indeed be a reasonably safe means of travel.

PERRY A. FELLOWS,* M. AM. SOC. C. E. (by letter).†—This paper emphasizes some of the points that the writer has stated‡ on other occasions. The methods outlined have been used in a number of instances as an aid in selection of airport sites.§

There are two limitations that should be definitely borne in mind whenever an attempt is made to codify characteristics: First, such a method is a tool and should be honestly used for the reason that, just as with any other tool in the hands of those who so wish, it can be misused; and, second, the group best suited to make use of such a tool is the engineering group.

If these two principles are kept uppermost in mind the use of airport characteristics will avoid the criticism that may be otherwise met. There are people who oppose setting up such a classification as that recommended in the paper;‖ especially do they oppose it on the grounds that it is possible for some one to believe that a way is thus made easy for the layman to select the

* City Engr., Detroit, Mich.

† Received by the Secretary, March 13, 1930.

‡ *American City Magazine*, May, 1928, and a more complete discussion in the Associated Technical Societies *Bulletin*, April, 1928, reprinted in *Engineering News-Record*, September, 1928.

§ "Standards and Score Card for Municipal Airports," by N. L. Engelhardt, Jr., *Airway Age*, October, November and December, 1928.

‖ *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2308.

best possible airport site. Such criticism can always be met with the cautionary advice that the instrument should be used only in its proper field and that its use should be entrusted only to the competent and qualified engineer.

W. W. CROSBY,* M. AM. SOC. C. E. (by letter).†—To one who has been familiar with the development of County, State, and Federal Highway systems during the past thirty years or more, this paper has suggested some analogies of interest. Both airways, with their airports, and highways, with their foci or termini, are transportation systems. The locomotion may seem quite different, but certain principles are common, and airships do "land" even though, as the author suggests, air and water navigation have much in common.

When the highway systems were first started, every little village fought strenuously to have the improved roads run through it and along its main street. As the expansion of the systems took place, and such transcontinental routes as the "Lincoln Highway" were conceived, officials in the larger cities also fought for the routes to run through their territory.

A few years ago a reaction took place. The same places began to appeal piteously to the highway authorities for the removal of the through traffic to by-passing routes so that the localities might be relieved from the serious dangers and damages caused to the local use of the streets by the through traffic; and a general movement along this line is already under way.

It seems to the writer that analogies accompany these facts; and he believes that they should be seriously considered in connection with the problems of airport location.

In this same connection a detail of airport construction may have some weight. May not the developments in airport surfacing have some effect on location just as the developments in highway surfaces have affected highway location?

Formerly, many conflicting factors influenced the type of surface necessary on highways. For example, smooth, hard surfaces were not suitable for slow, horse-drawn traffic and vehicles with metal tires that ground the road surfaces into dust. As the motor traffic has become predominant on many systems of highways it has simplified the problems of surfacing by removing these conflicting factors, so that the traffic is composed only of soft-tired vehicles which require smoothness and which create no detritus. Curves are required to be flatter, visibilities must be longer, and grades may be steeper; but alignment should be more direct and widths must be greater. To give general satisfaction surfaces may be less varied and more "standard".

May not some of the developments in air vehicles, that now seem quite probable to threaten, similarly affect the decisions as to airport construction and location? For instance, suppose that a mechanical modification in the wing of a plane should be developed so that a plane could check its horizontal speed and drop gently, and almost vertically, to its landing. Would not the allowance for "gliding angle" in airport location be reduced? Similarly, if the airplane could be checked in its forward movement, would not the necessity

* Cons. Engr., Coronado, Calif.

† Received by the Secretary, April 16, 1930.

of depending on the friction of the tires or the tail-skid for this result be relieved so that the surfacing of the landing field could be simplified and improved? Possibly, also, the rising angle of a starting plane could again be increased, and the location again affected. These results might affect the size of the area required and thus enable locations to be made advantageously where now they seem impossible.

Furthermore, if an airport may be regarded as something distinct from an airfield, may not the location problems of the former be simplified? Why should not the housing, repairing, fueling, oiling, tuning-up, and such work go on in convenient but separated "yards" or "docks" (as in the case of railroads and shipping lines), so that the ports or depots might be located with the convenience to the passengers that is necessary for commercial popularity?

Certainly training or practice activities are not necessary functions of airports.

Even in the new art of airway engineering, it seems to the writer, after considering the many mistakes that have been proved on him as well as on others in years of highway engineering, the saying that "the wise man learns from the experience of others; any one can learn from his own", is still worth remembering.

E. K. Smith,* Assoc. M. Am. Soc. C. E. (by letter).†—The considerable increase in air passenger traffic, quite largely due to the lowered passenger rates, and the increase expected in express service should cause special importance to be placed on certain of the factors in airport location covered in Mr. Baker's paper. These are, particularly, the physical features, the accessibility to the community served, and the ground transport facilities to and from the airport. For mail service alone facilities for transport to and from the airport will not be so important. It is probable that within a few years all major airports will have pneumatic tube service from the main post office.

For a purely express or mail business the surroundings, appearance of the field, etc., do not play as important a part as for passenger traffic. Air transportation offers to-day a specialized service for passenger transportation already realized and seeing each year fast improvement in air equipment. Nevertheless, as stated by C. M. Keys, President of the Transcontinental Air Transport Company:‡

"Only 10% of aviation is in the air; 90% of it is on the ground.

"As regards passenger carrying on a transcontinental scale, that is probably an under-statement; the ratio between ground preparation and actual flight is likely to be wider than that."

For all three classes of performance (mail, express, and passenger), air transportation offers a highly specialized service, superior in speed to any other form. While it is yet seriously handicapped by unfavorable weather conditions, the most unfavorable weather also handicaps highway and railroad service, sometimes for a much longer time.

* Asst. Mgr., Highways and Municipal Bureau, Portland Cement Assoc., Chicago, Ill.

† Received by the Secretary, April 21, 1930.

‡ *Chicago Herald and Examiner*, Interview, November 4, 1928.

To build up the general public use of air service, it is already evident that greater attention must be paid to permanence and stability of construction on carefully designed airports, with special attention to the appearance of the airport and the provision of comfort for passengers. The public is already educated to expect and secure comforts and conveniences at railroad stations and bus terminals that were unheard of a few years ago. Without reasonable provision for corresponding conveniences at airports proper development of air transportation cannot be expected to continue steadily at the desired rate. Even in the appearance of the field, the buildings should not be mere sheds or brick barns, but should give, without undue expense, the appearance of careful design and stability suitable for an established industry giving important daily service to the public. Hotels, restaurants, rest rooms, etc., are already making their appearance at American airports as they have for several years past in Europe. Certainly it is reasonable to expect a just proportion of this cost as a proper operating cost of the field to be returned from the field charges for service.

Therefore, in locating the airport, consideration must be given to the ease with which the physical features may be adapted to providing facilities for the comfort and convenience of air passengers. It is also evident that for a number of years to come, the fields will be visited by thousands of people to see shows, races, etc., or merely to observe the daily business of the field. This has already been capitalized at European airports where amusement and athletic parks frequently adjoin the field and where grand stands are frequently supplied for the use of spectators. A voluntary attendance of many thousand people at any municipal air field is an advertising asset far more important than that which can be secured by thousands of dollars worth of paid newspaper and magazine publicity. The impression carried away by these people and passed on by them to their friends will be of the highest importance.

A location for the airport that makes it easily accessible to visitors, a site large enough to provide ample parking space in addition to the bare necessities for planes, and, if practicable, a location and surroundings that present an attractive appearance, are features of real value to the community. Provision of rest rooms and observation stands will encourage the public support of the airport. These are facilities freely provided in any park or other place of public gathering. The airport, to-day, is in many respects more important than the public park, or it should be made so. Paved sidewalks and paved drives and aprons around the hangars are equally important as effecting the comfort of visitors and patrons and keeping them free from mud and dirt.

It has been suggested that present condition of the ground surface will affect the choice of an airport location. This is especially true in so far as it affects the drainage required, etc. It is certain that paved runways for landing and take-off will shortly be a necessity at most of the important airports. A turf or light surface satisfactory a few years ago for the use of light planes will no longer serve the hundreds of heavy transport planes coming into use.

A tabulation has been published* which shows twenty-seven standard models of airplanes weighing more than 5 000 lb. A considerable number in

* *Aviation*, January 18, 1930.

transport service to-day weigh more than 10 000 lb. and the Boeing Air Transport Company and the Western Air Express are placing planes in long-distance transport service weighing, respectively, 17 500 and 22 000 lb.

The U. S. Department of Commerce, as a result of tests, states that the landing load should be increased 100% to allow for impact. This brings a wheel load on the surface of the field ranging from 5 000 to 22 500 lb.

Another factor requiring paved runways for safety and economy is the reduced distance required for take-off. On its flight to America, with complete load of fuel, the *Bremen* required a starting run of only 1 800 ft. on the Portland cement concrete runways at Dessau, Germany. At Baldonell, in Ireland, where the Atlantic flight was actually started, 4 750 ft. were required for take-off from the runways of rolled cinders and ashes in spite of the smaller load on the ship.

Despite the shortage of money for building purposes in Germany, concrete runways have been constructed during the three years since 1927 at Dessau, at the Halle-Leipzig Airport, at the Travemünde Airport, Frankfurt-am-Main, and at the Giessen Airport. In this country, concrete runways similar to the highest type of highways have been constructed at the New York City and Detroit Municipal Fields, at the Curtiss-Steinberg Field, at East St. Louis, Ill., and a number of others. It is evident that the character of the present field surface no longer plays as important a part as formerly in determining airport location since for any important airport a paved surface will be provided to support the heaviest planes, reduce the length of take-off, and greatly reduce the maintenance required on the remainder of the field.

Donald M. Baker,* M. Am. Soc. C. E. (by letter).†—The effort was made in the preparation of this paper to emphasize the relationship of an airport to the entire city or regional plan, and also to set up certain fundamental requirements as a guide for the use of those who are responsible for the selection of sites for airports. The system of rating was suggested primarily as a mechanical device whereby various sites which might be offered for consideration could be compared, one with another. Whenever this method is used as a basis of comparison, the actual items specified are not so important, provided some judgment is used in selecting them, as the method whereby they are applied and the fact that they must be applied uniformly to all subjects being judged. Without question, each of the ten points‡ suggested by the writer could have been subdivided into from two or three to a dozen sub-headings, each of which in itself could have been given a proper weight. This system was used by the writer early in 1928.

The paper referred to by Mr. Fellows§ illustrates the detail to which such systems of rating may be carried. It was not felt necessary to go into such detail in the writer's paper, as it was thought that any one using the method could, with better results, expand the ten points given. The weights suggested for each factor were suggestive only and might be varied, depending on the

* Pres., Board of City Planning Commrs.; Cons. Engr., Los Angeles, Calif.

† Received by the Secretary, May 14, 1930.

‡ *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2309.

§ See footnote, p. 1398.

different uses to which the field to be selected was put, the main point being that all sites should be compared by the same measuring stick.

The writer does not agree with Mr. Nolen's suggestion* as to the elimination of the factor of legislative control. Unless all sites under consideration are within the limits of an incorporated city where zoning and police regulation exist, it is very essential that some measure of control be placed over the development surrounding the airport, in order that height of buildings, use of property, development of highways, etc., may be regulated. If the site lies within unincorporated territory, proper co-operation on the part of county officials may affect this, but if this co-operation cannot be obtained it constitutes a serious drawback.

Accessibility, which should be expressed in terms of time, and not in terms of distance, is primarily an economic factor that can be evaluated in most cases in terms of dollars and cents. In a large metropolitan center, secondary airports will become absolutely essential for private planes used primarily in short hops. These secondary sites, however, can be made relatively small in size, but should be located, of course, so as to be accessible to the better class residential sections of the city wherein will reside most of those using this type of air transportation. The writer is still convinced, however, that the ultimate commercial load in air travel will be the long haul, in which case accessibility to the heart of the city, if maintained within reasonable limits, is not so important. Short-haul passenger traffic has, of course, been developed recently to a considerable degree, but, until night flying comes into existence as a regular feature, passenger travel between cities 400 to 600 miles apart, will not show much saving of time during business hours. At present, traveling can be done in such cases by rail by leaving the office after closing hours, arriving at one's destination the next morning, having a full business day, and returning the following night, whereas travel by air made during daylight hours does not show any comparable saving of business hours. The gradual reduction in fares for aerial transportation, however, is increasing its popularity. The distance which a major traffic air terminal is located from the heart of the city in the final analysis becomes a question of adjusting capitalized cost of added distance from the business center against the cost of land necessary for the site, and arriving at a minimum total cost. Land cost varies inversely with the distance from the center of the community, and 5 min. in time distance may mean a vast sum in land investment.

Rapid transit lines are likely to serve the transportation needs of an airport better than highways, although any site should be located so that large crowds may have ready access to it on important occasions. The question of space necessary for landing and take-off areas is one that is still open to discussion. In 1928 much discussion was extant as to the probability of reduced landing and take-off speeds and consequent size of landing and take-off areas, but nothing has appeared during the intervening two years which would indicate that any smaller areas can be used satisfactorily. Ships are getting larger, and more of them are coming into use, resulting in greater crowding

* *Proceedings*, Am. Soc. C. E., April, 1930, Papers and Discussions, p. 860.

of airports. This point also must be kept in mind—that if in the future excess land has been acquired for an airport site, it can always be disposed of at a profit or used to good advantage for park or recreational purposes. The basis of rating given was primarily to apply to the sites to be used and not to the finished field.

The writer agrees with Mr. Angell* in his statement as to the desirability of having surface fields, and also with Mr. Marsh† as to the need for recreational facilities. This is one situation which can be far better cared for at publicly owned than at privately owned airports. In the latter every possible acre of ground must be made to pay revenue, and a private owner cannot afford to devote a large space to recreational facilities or allow excessively large areas for automobile parking.

The writer heartily agrees with Mr. Nolen's‡ statement that an airport is essentially an industrial establishment, not only through the attendant activities and development which will surround it, but by virtue of the noise, crowds, dust, etc., which must exist even at the best regulated port.

Mr. Grimm's§ discussion and the re-arrangement of governing factors brings out a point the writer had in mind. The factors must be given different weights when applied to fields used for different purposes. Accessibility and transportation are far more important in a small field used by owners of private planes for business or pleasure purposes than in the case of a major terminal, and, at the former, meteorology is not so important since departures and landings are not usually made on schedule. The factors given by Mr. St. John‖ illustrate the same thing, although the writer believes that probably too much weight is given by him to meteorology, which totals 30 points in Factors 1, 2, and 6. It is not thought that Item No. 13 in Table 1‖ is necessary. The airplane factory sites should come to the airport and the port should not go to the factory. His Item No. 7, "Prospective Neighborhood Development", would be one sub-factor under the writer's factor, "Legislative Control."

The writer is very familiar with the excellent work being done by the Los Angeles County Regional Planning Commission in the development of a regional plan of airports in that county, which is referred to by Mr. Fox.¶ The latter's discussion brings out very admirably the method and study which was suggested by the writer as a preliminary to the selection of any site or sites, and emphasizes the necessity of a comprehensive airport plan for any region.

Aeronautical transportation is here to stay, and the sooner the fact is accepted by the general public, including officials, the sooner they will realize that a regional plan of airports is just as essential as a regional plan of highways, and the more money will be saved in the ultimate acquisition of such sites. Mr. Fox's illustration** of earning ability of Air Terminal No. 8, in the

* *Proceedings*, Am. Soc. C. E., April, 1930, Papers and Discussions, p. 855.

† *Loc. cit.*, p. 857.

‡ *Loc. cit.*, p. 860.

§ *Loc. cit.*, p. 861.

‖ *Loc. cit.*, p. 864.

¶ See p. 1389.

** See p. 1393.

San Gabriel Valley, California, is very apt. It shows the proper economic approach to the matter and develops the fact that in the future airports can undoubtedly be classed as publicly owned utilities of a self-supporting nature.

The writer is much gratified with the general discussion which the paper has brought out, and hopes that the importance of airports in the general city or regional plan has been emphasized. Private companies, as well as public agencies, should realize that the location of an airport is a major factor making for its success, both from the standpoint of the port itself and of its service to the community. Much has been learned by railroads through experience in improper location of terminals and subsequent re-arrangements. No site should be chosen by any community until its location has been given the most thorough study.

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PAPERS AND DISCUSSIONS

EFFECT OF TURBULENCE ON THE REGISTRATION OF CURRENT METERS

Discussion*

By H. R. Leach, M. Am. Soc. C. E.

H. R. Leach,† M. Am. Soc. C. E. (by letter).‡—This paper has re-opened the subject of the performance of current meters in turbulent water, a question that has never been satisfactorily settled. It has generally been recognized that, except under conditions closely approaching stream-line flow, current meters of all types may indicate velocities departing more or less from the true velocity of the current, and that as conditions depart further from stream-line motion and turbulence increases, the departures of the indicated from the true velocities become increasingly greater. During the thirty years preceding 1930 several investigators have made excellent observations to determine the magnitude of these departures, and although definite quantitative results have been obtained there has been little, if any, attempt to interpret them so that velocities observed under abnormal conditions could be modified to give more nearly correct results.

In performing these experiments the authors show their recognition of the futility of trying to dispose of a troublesome question by saying, as has been done by many writers on the subject, that current meters should not be used in turbulent water. This merely "sidesteps" a situation that frequently confronts the man engaged in stream measurement, who often has no choice but to use his meter in a turbulent stream or to give up the measurement altogether. Even an admittedly inaccurate measurement may be of value, particularly if some idea of the probable error can be obtained.

In attempting to solve this problem the authors first ran a series of experiments on meters in turbulent water, but found too many factors involved to permit a clear interpretation of the results. Therefore, "rather than leave

* Discussion of the paper by David L. Yarnell and Floyd A. Nagler, Members, Am. Soc. C. E., continued from May, 1930, *Proceedings*.

† Prin. Asst. with Robert E. Horton, Voorheesville, N. Y.

‡ Received by the Secretary, May 6, 1930.

the matter in this incomplete and unsatisfactory state, it appeared desirable to attempt to resolve the problem into simpler elements and to attack them separately."* Accordingly, they performed two other series of experiments. They did not, however, follow up the results obtained in the later experiments and attempt to interpret the performance of the meters in turbulent water in the light of the results found in their subsequent work. This is the only criticism that the writer makes of an otherwise excellent account of well planned and carefully performed experiments. It is to be hoped that the discussions of this paper and the authors' closure will analyze these experiments and suggest some rational method whereby current-meter measurements, that must of necessity be made in turbulent water, can be corrected to give a fairly close value of the true discharge.

As far as the writer is aware, B. F. Groat, M. Am. Soc. C. E., is the only investigator who has suggested a definite method of correcting current-meter measurements made in turbulent water.† In connection with measurements of the discharge through turbines at Massena, N. Y., he used Price and Haskell meters and suggested that the true velocity be taken as that indicated by the Haskell meter plus one-seventh of the difference of the velocities indicated by the Price and the Haskell meters.

Experiments on Current Meters, by B. F. Groat.—A few years after he had made this suggestion, Mr. Groat ran a series of tests in connection with a comparison of current-meter measurements with chemical gauging.‡ These covered a wide range of conditions. The writer has recomputed these experiments and the results are presented in this discussion to show to what extent they corroborate the authors' experiments.

The tests were made in the Naval Tank of the University of Michigan on a small Price meter, two low-pitch Haskell meters, and two Ott meters. The meters, with tails removed, were rigidly attached to arms on the rating car with their axes parallel to the axis of the tank and to the direction of motion of the car. The arms were arranged so that they could be oscillated in three directions, that is, vertically, horizontally at right angles to the axis of the tank, and longitudinally parallel to the axis of the tank. They were connected so that the same movement was imparted to each of the five meters. Their axes were always parallel to the direction of motion of the car except that in the longitudinal oscillations the arms gave the meters a slight rocking motion in a vertical plane. The manner in which the frame was oscillated was not stated. Mr. Groat plotted the observed revolutions per foot for different oscillations against the velocity of the car.

By first constructing a rating curve from the normal observations and then determining the indicated velocity for each observation, the writer obtained the difference between the indicated velocity and the forward velocity of the rating car. The rate of oscillation and the velocity of the car were added vectorially to obtain the absolute velocity of the water relative to the meter

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2625.

† *Transactions*, Am. Soc. C. E., Vol. LXXVI (1913), p. 819.

‡ "Chemi-Hydrometry and Its Application to the Precise Testing of Hydro-Electric Generators," *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 1231, Pt. V.

and the angle, α, between the direction of the absolute velocity and the axis of the meter. This recomputation was made only for the observations on the Price and Haskell meters.

Small Price Meter: Horizontal Oscillations.—The observations for horizontal transverse oscillations are plotted on Fig. 23. The indicated velocity in percentage of the cosine velocity is plotted against α, the angle the tangent of which is the velocity of the horizontal oscillation, divided by the velocity of the car. The cosine velocity is the absolute velocity times cos α, and is, of course, the velocity of the car. Curve *A* is the absolute velocity of the water relative to the meter in percentage of the cosine velocity.

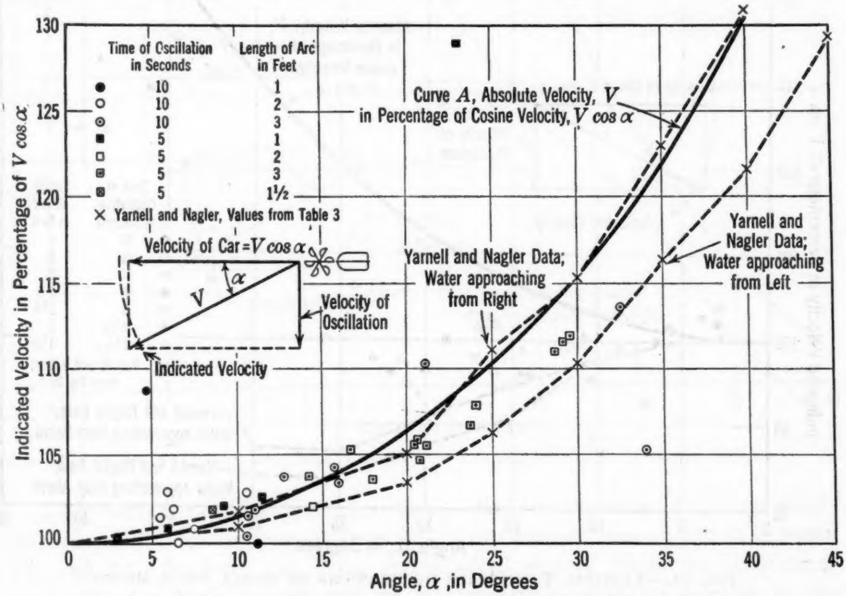

FIG. 23.—HORIZONTAL TRANSVERSE OSCILLATIONS ON SMALL PRICE METER.

Were it not for the interference caused by the yoke supporting the bearings of the cups, the Price meter apparently would record the absolute velocity and, therefore, would over-register with respect to the cosine velocity. Up to angles of about 15° it does this. For angles greater than 15° the indicated velocity is a few per cent. less than the absolute velocity owing probably to the interference of the yoke.

The values presented by Mr. Groat are in good agreement with the average of the Yarnell and Nagler values for right- and left-hand inclinations. This would be expected from the fact that the oscillating movement would give the mean of the right-hand and left-hand effects. Although the Iowa experiments show that the ratio of indicated to cosine velocity depends to some extent on the velocity, this does not appear to have affected Mr. Groat's results appreciably.

Small Price Meter: Vertical Oscillations.—The observations for vertical oscillations are plotted on Fig. 24. Within the range, $\alpha = 0°$ to 25°, they

show an indicated velocity slightly less than the cosine velocity. For angles greater than 25° the indicated velocities are greater than the cosine velocities. This does not agree with the results found by the authors because, in all cases up to the limit of their experiments, the indicated velocities were less than the cosine velocities. Experiments made by the U. S. Geological Survey in 1898,* on one of the earlier type Price meters gave observed velocities in all cases greater than the cosine velocities. On the other hand, the results found by the authors fall between values observed by Brown and Forrest Nagler,† M. Am. Soc. C. E.

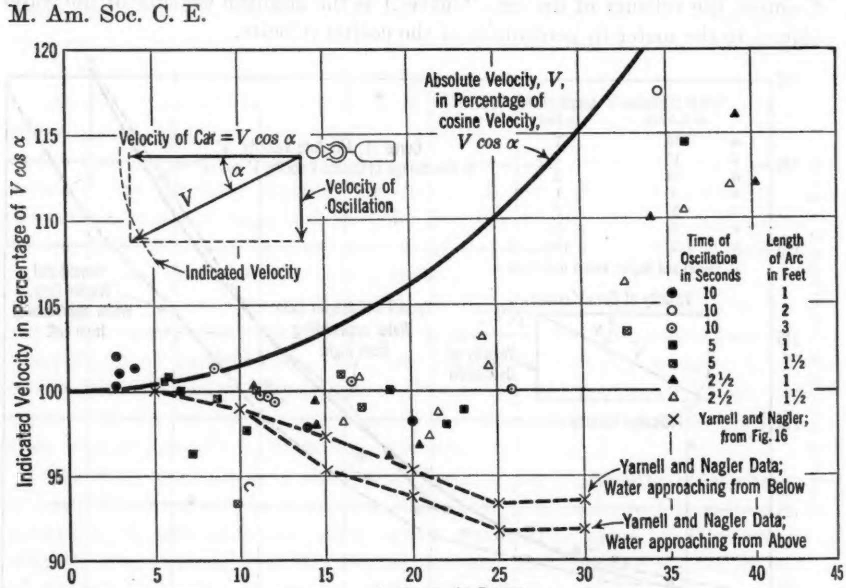

FIG. 24.—VERTICAL TRANSVERSE OSCILLATIONS ON SMALL PRICE METER.

The difference may be due partly to the fact that in one case the meters were held at a constant angle of inclination against the current, while in the other they were oscillated, probably at a varying rate. It may be noted that the authors' results on the acoustic Price meter do not show as great departures from the cosine velocity as the electric Price and are in good agreement with Mr. Groat's observations up to an angle of about 20 degrees. Between angles of 20° and 30° the acoustic meter gave indicated velocities about 3% less than the cosine velocity.

Haskell Meter: Horizontal and Vertical Oscillations.—The observations for the Haskell Meter No. 1 as recorded by Mr. Groat,‡ are plotted on Fig. 25. The observations for both horizontal and vertical oscillations are given, and it can be seen that there is no marked difference between the two, although the latter may have slightly smaller departures. Mr. Groat's results agree

* Nineteenth Annual Rept. U. S. Geological Survey, Pt. IV, p. 30.

† "Stream Gaging," Liddell, First Edition, 1927, p. 144.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXVI (1916), p. 1246.

well with the departures observed at Iowa for water approaching the meter horizontally from the left-hand side; but they are greater than the departures listed by the authors for water approaching from the right side.

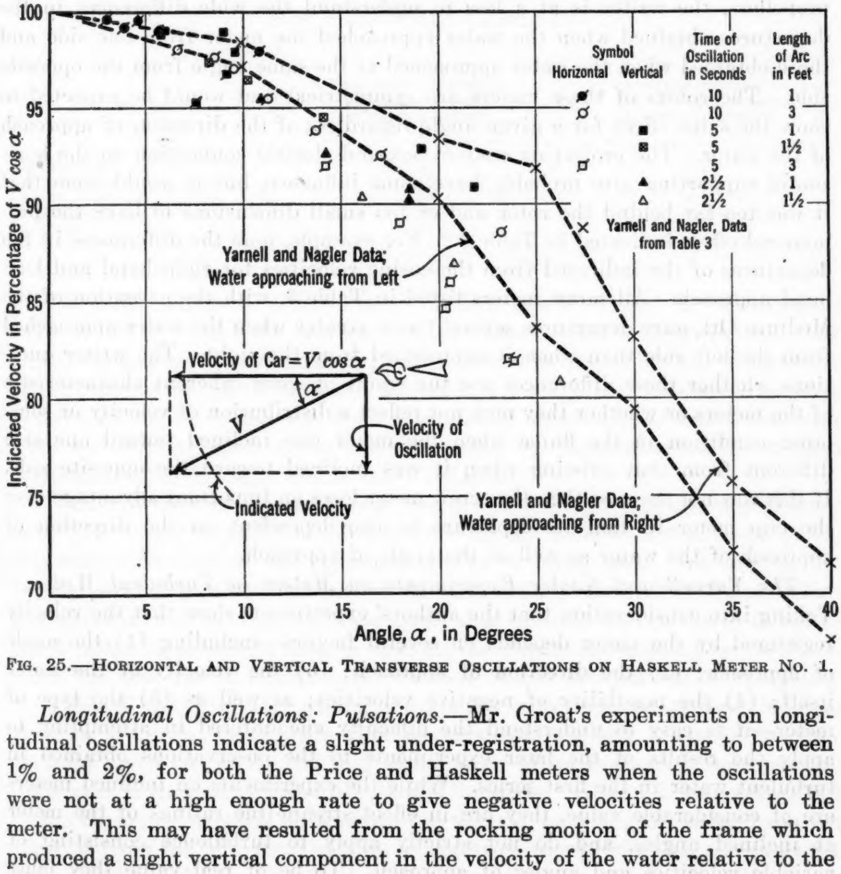

FIG. 25.—HORIZONTAL AND VERTICAL TRANSVERSE OSCILLATIONS ON HASKELL METER NO. 1.

Longitudinal Oscillations: Pulsations.—Mr. Groat's experiments on longitudinal oscillations indicate a slight under-registration, amounting to between 1% and 2%, for both the Price and Haskell meters when the oscillations were not at a high enough rate to give negative velocities relative to the meter. This may have resulted from the rocking motion of the frame which produced a slight vertical component in the velocity of the water relative to the meter. When negative velocities were created, over-registration occurred, as was the case in the authors' experiments, and probably for the same reason, namely, as a result of contacts when the rotor was running backward.

The Yarnell and Nagler Experiments on Inclined Meters.—In general, Mr. Groat's observations agree fairly well with these obtained by Messrs. Yarnell and Nagler. The writer is inclined to believe that a part, at least, of the differences between the two, result from the fact that in the authors' experiments a steady flow impinged against a meter held at a constant angle, while in Mr. Groat's experiments the oscillations probably caused both a varying angle and varying velocity. For this reason the latter may be somewhat more representative of actual conditions in turbulent water. The departure resulting from a constant angle of impact is not the same as the average departure resulting from a varying angle having the average value of the

constant angle. Furthermore, it is difficult to account for some of the results obtained by the authors.

In the case of the experiments on the meters with horizontally inclined propellers, the writer is at a loss to understand the wide differences in the departures obtained when the water approached the meter from one side and those obtained when the water approached at the same angle from the opposite side. The rotors of these meters are symmetrical and would be expected to show the same effect for a given angle regardless of the direction of approach of the water. The projecting contact box and electric connection on the horizontal supporting arm probably have some influence, but it would seem that it was too far behind the rotor and of too small dimensions to have the pronounced effect indicated in Table 3.* For example, note the differences in the departures of the indicated from the cosine velocities for right-hand and left-hand approach. All screw meters listed in Table 3, with the exception of the Medium Ott, gave departures several times greater when the water approached from the left side than when it approached from the right. The writer questions whether these differences are the result of some inherent characteristic of the meters or whether they may not reflect a distribution of velocity or some other condition in the flume when the meter was inclined toward one side different from that existing when it was inclined toward the opposite side. If this was not the case then the screw meter loses an important advantage over the cup meter in that its departure is also dependent on the direction of approach of the water as well as the angle of approach.

The Yarnell and Nagler Experiments on Meters in Turbulent Water.—Taking into consideration that the authors' experiments show that the velocity registered by the meter depends on several factors—including (1) the angle of approach; (2) the direction of approach; (3) the velocity of the water itself; (4) the possibility of negative velocities; as well as (5) the type of meter—it is easy to understand the difficulty encountered in attempting to apply the results of the later experiments to the observations obtained in turbulent water in the first series. While the experiments on inclined meters are of considerable value, they are in effect stream-line ratings of the meter at inclined angles, and do not strictly apply to turbulence consisting of variable velocities and angles of approach. To be of real value they must explain the performance of the meter in turbulent flow.

It is unfortunate that in the turbulence experiments the observations were not taken over a longer period so that the performance of each meter at a given point could be compared. In Experiments Nos. 8 and 9, in which "duplicate readings as shown in Table 1 were taken at each point in the cross-section in order to determine the possibility of error by taking observations for too short a time in turbulent water,"† it was found that "individual observations of velocity in this turbulent water, taken for the usual length of time, may vary as much as 10 to 20% * * *." Such errors, however, are largely compensating. In Experiments Nos. 8 and 8(*a*) (Table 1‡) the average differ-

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2631.

† *Loc. cit.*, p. 2634.

‡ *Loc. cit.*, p. 2624.

ence between duplicate readings at each of the eight points, as well as the maximum difference at any one of the points, is given in Table 7.

TABLE 7.—COMPARISON OF VELOCITIES REGISTERED IN DUPLICATE EXPERIMENTS NOS. 8 AND 8(a). (TABLE 1.)

Meter.	Maximum difference at single point, in feet per second.	Average difference for all eight points, in feet per second.
Electric Price	0.28	0.03
Acoustic Price	0.31	0.05
4-blade Hoff	0.46	0.03
Small Ott	0.22	0.12
Large Ott	0.31	0.02

The large maximum differences at single points show that the observations were too short to get the correct average value of the velocity indicated by any of the meters at any one point, and this renders impossible any comparison of the observations of the different meters at a given point. The small average differences, however, indicate the compensating nature of the errors resulting from too short an observation, and this makes the comparison of the average performance at the eight points more reliable.

Although the authors ran Series Nos. 2 and 3 for the purpose of obtaining data that would explain the performance of the meters in turbulent water, this cannot be done by the experiments thus far presented. Not only do the later experiments fail to account for the observations obtained in the turbulent flow experiments, but they indicate either that there is some hitherto unrecognized factor in turbulent flow, or that all the conditions existing in the flume when the flow was turbulent have not been clearly brought out in the paper.

Table 3 and Fig. 15,* giving the results with inclined meters, indicate that, in all cases, the 4-blade Hoff meter should register a smaller velocity than the small Ott meter and that as the angle of inclination, α, increases the difference in indicated velocities should increase. Although the magnitude of the departure depends on whether the water approaches the meter from the right or from the left, the direction of approach was presumably the same for both meters, and they should show the same relative departures. Moreover, the authors conclude that if, as was the case in Series No. 1, the electric contacts are made at intervals of several revolutions, all meters will register the correct average of a variable velocity. When, however, the average of the duplicate readings of the small Ott and the 4-blade Hoff meters in Experiments Nos. 8 and 8(a) are compared, it is found that at four of the eight points the Hoff meter gave velocities from 4 to 17% greater than the small Ott meter. In Experiments Nos. 9 and 9(a) the Hoff meter indicated velocities from 12 to 42% greater than the small Ott. This is not explained by insufficient duration of observation or by anything shown by the oblique flow experiments.

The unusual condition of turbulence existing in the flume is shown by the fact that in three out of the eight experiments (two of the total of ten being

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2633.

duplicates) the screw meters registered from 25 to 50% greater velocity than the mean derived from the discharge over the standard weir. Considering that screw meters always under-register oblique velocities, it must be concluded that there was an area of zero or negative flow in the section and that the effective area of forward flowing water was materially less than the total cross-sectional area and therefore resulted in abnormally high velocities. While areas of stagnant or backward flowing water are sometimes encountered in stream-gauging work, particularly immediately behind obstructions, it is generally possible to recognize and to take this condition into account, and such highly abnormal conditions as were apparently created in the authors' flume would be almost unique in field work.

Correction of Observations Made in Turbulent Water.—The writer believes that notwithstanding the impracticability of attempting to obtain the proper correction to current meter observation by evaluating each of the several factors influencing the registration of the meter, it is possible to carry out field measurements in such a manner as to reduce, if not entirely eliminate, the errors introduced by turbulent flow. There are several methods that should be given consideration by any one who must measure a stream that is more or less turbulent.

Two Measuring Sections.—If two measuring sections are close enough together so that there is no change in the quantity of water passing the sections and yet far enough apart so that the conditions at the sections are different, some indication is given of the amount of turbulence in the stream and also which of the two will give the best result. This is increasingly valuable if the shapes of the sections and the distribution of velocities are materially different. This, of course, does little more than show which of the two sections is the better measuring section. Where, however, measurements must ordinarily be made at an unfavorable section—for example, from a bridge supported by several piers—check measurements made from a boat or from a temporary cable a short distance up stream will give the correction to be applied to measurements made at the lower section.

Elimination of Turbulence by Baffles.—This is obviously the most satisfactory method of dealing with turbulence although it is necessarily confined to use in a stream of small dimensions. If, however, a number of measurements are to be made and accuracy is of more than usual importance, it may be the least expensive of the methods considered.

Measurement of Turbulence.—It seems to the writer that a rather simple device could be constructed that would give the horizontal and vertical directions of the current at each point of observation. Knowing these directions, the correction to be applied to the registered velocity could be made by means of the departures shown on diagrams similar to those presented by the authors. The use of such an instrument would be confined to shallow channels or to streams having a low velocity.

Inclination of Meters.—In the case of rigidly suspended meters it would seem possible to arrange the suspension so that the instrument could be rotated in both horizontal and vertical planes through 15 or 20° from its normal position perpendicular to the section. In this way the direction giving the greatest

velocity could be found by trial, and the velocity at the point of observation corrected accordingly. This method could not be used with a cup meter as moderate changes in its horizontal position would not change the velocity indicated. The disadvantage of this method, aside from the trouble of providing the special suspension, lies in the amount of time required at each point. Instead of a single observation of usual duration, four or five observations would be required and these should be for sufficient length of time to get the true average registration and eliminate the effect of differences resulting from pulsating flows.

Use of Two Meters.—If simultaneous measurements are made with two instruments, one of which is a cup meter and the other a screw meter, it may be assumed with considerable reliability that the true discharge lies between the flows indicated by the two meters. Unless the turbulence is caused almost entirely by vertical components of velocity, the cup meter will over-register. The screw meter will always under-register. It remains to determine at what point between the velocities indicated by the two meters the true velocity lies.

That this method merits more than passing attention is shown by the results it gives when applied to the highly abnormal turbulence existing in the authors' flume. Table 2* gives the departures of the velocities indicated by the different meters from the true mean velocity in the flume. Fig. 17† gives the relative registration of the meters when it is assumed that the current approaches the meter from all directions with equal frequency, and lacking any more specific data, it can be assumed that this was the condition in the flume. In general, the curves in this diagram indicate that:

(*a*) Under-registration of the 4-blade Hoff is four times as great as over-registration of Price meter.
(*b*) Under-registration of the medium Ott meter is twice as great as over-registration of Price meter.
(*c*) Under-registration of the small Ott meter is equal to over-registration of Price meter.

Therefore, in order to get a closer approximation to the true velocity than is given by any of the meters alone:

(1) Deduct one-fifth the difference between the Price and the 4-blade Hoff velocities from the Price velocity.
(2) Deduct one-third the difference between the Price and the medium Ott velocities from the Price velocity.
(3) Deduct one-half the difference between the Price and the small Ott velocities from the Price velocity.

The error in the computed velocities using this method is given in Table 8.

It is obvious that for even such highly abnormal turbulence as there was in the flume used for those experiments, and with nothing to indicate the predominating direction of approach, or the nature of the turbulence, this method gives much better results than were obtained by any single meter. Omitting Experiment No. 4, more than one-half the results with the two-meter method are in error by less than 2% and the greatest error is 6.2 per cent. With the single meters more than one-half the results are more than

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2625.
† *Loc. cit.*, p. 2635.

13% in error and the greatest error is 61.6 per cent. In all cases except one, the two-meter method gives a better result than any of the single meters.

In Experiment No. 4 the electric Price meter indicated a lower velocity than either the large or small Ott meters. It under-registered by 17.4%, while the acoustic Price over-registered by 18.5%; there is, therefore, some question concerning this run. Experiments Nos. 2, 5, 7 and 10 were omitted in the preceding comparison for the reason that in all but one the screw meters indicated from 25 to 50% greater velocity than the true velocity, and in Experiment No. 7 three of the four screw meters over-registered. As has already been pointed out, this indicates a condition that is not entirely explained by the data presented.

TABLE 8.—ERROR IN VELOCITY COMPUTED FROM VELOCITIES REGISTERED BY TWO METERS.

Experiment No.	Small Price and 4-Blade Hoff.			Small Price and Medium Ott.			Small Price and Small Ott.		
	Departure, in percentage.	One-fifth the difference, in percentage.	Error in computed velocity, in percentage.	Departure, in percentage.	One-third the difference, in percentage.	Error in computed velocity, in percentage.	Departure, in percentage.	One-half the difference, in percentage.	Error in computed velocity, in percentage.
3	+18.9*			+18.9			+18.9		
	−61.6†			−28.3			−17.4		
	80.5‡	6.1	+ 2.8	47.2	15.7	+ 3.2	36.3	18.1	+ 0.8
4	−17.4			−17.4			−17.4		
	−20.2			−19.9			−15.8		
	2.8	0.5	−17.9	2.5	0.8	−18.2	1.6	0.8	−16.6
8	+12.8			+12.8			+12.8		
	−29.5			−22.2			−12.6		
	42.3	8.5	+ 4.3	35.0	11.7	+ 1.1	25.4	12.7	+ 0.1
8(*a*)	+11.1			+11.1			+11.1		
	−13.5			−19.3			−12.9		
	24.6	4.9	+ 6.2	30.4	10.1	+ 1.0	24.0	12.0	− 0.9
9	+ 5.1			+ 5.1			+ 5.1		
	− 5.4			−13.5			− 3.1		
	10.5	2.1	+ 3.0	18.6	6.2	− 1.1	8.2	4.1	+ 1.0
9(*a*)	+ 9.5			+ 9.5			+ 9.5		
	−11.2			−13.9			− 1.3		
	20.7	4.1	+ 5.4	23.4	7.8	+ 1.7	10.8	5.4	+ 4.1

* Departure by Price meter, from authors' Table 2.
† Departure by screw meter, from authors' Table 2.
‡ Difference in departures.

Cable Suspension.—The preceding discussion relates to rigidly suspended meters whereas by far the greater number of current-meter measurements are made with cable suspension. Several of the methods described for handling

measurements in turbulent water will apply equally well to cable suspension. It should be noted in the case of the two-meter method that the screw meter will not under-register as much with cable suspension as when rigidly suspended owing to the tendency of all meters, including the screw types, to point into the current and so register the absolute velocity.

In view of the results thus far presented the writer would suggest that further work be done to determine completely the effect of direction of approach on the registration of the nearly symmetrical screw meters, and that this be done in a channel of relatively large cross-section to eliminate, in so far as possible, the obstruction caused by the meter itself. It would also seem that turbulent conditions more nearly representative of those actually encountered in practice could be obtained in the 10-ft. hydraulic canal, and that some rational method of correction could be developed by comparing the performance of different meters at the same point with each other and with their registration with oblique flow.

In closing, the writer wishes to commend the work of the authors in their attempt to unravel an intricate problem. He well recognizes the impossibility of foreseeing before embarking on an experimental program, all the various phases that will develop during the course of the work. It is to be hoped that the authors will find the opportunity to carry out further investigation of this difficult question.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

GENERAL SPECIFICATIONS FOR STEEL RAILWAY BRIDGES

PREPARED BY COMMITTEES FROM THE AMERICAN SOCIETY OF CIVIL ENGINEERS AND THE AMERICAN RAILWAY ENGINEERING ASSOCIATION

Discussion*

By Messrs. S. N. Mitra, C. C. Westfall, and R. F. Patterson.

S. N. Mitra,† Jun. Am. Soc. C. E. (by letter).‡—The writer believes that the Committee was quite justified in increasing the allowable stress in steel by 50% for dead load (Article 300§). It is really in keeping with progress to admit that in this way engineers are learning more about steel structures. It is true that a decrease in the "factor of safety" is thus permitted, but a relatively high factor of safety does not seem necessary for dead load. The factor of safety has been justly termed a "factor of ignorance" and should not be justified if it can be shown that it is much less for dead load than for live load; that is, a designer is more certain of the value of the dead load than of the live load.

Occasionally, a bridge may be required to withstand unexpected shocks or strains for which it was not designed; but it could scarcely be expected that such occasional over-stresses would ever be produced by its own dead load. If at all, they would have to be produced by some external force which may well be considered in the same category as live load. Therefore, would it not be logical to assume that even if a bridge were designed with a marginal factor of zero for dead load and 3½ for live load, it could easily withstand 3½ times the normal maximum live load before it would fail? Would it not follow

* Discussion of the General Specifications for Steel Railway Bridges, continued from May, 1930, *Proceedings*.

† Asst. Bridge Engr., Calcutta Branch, Rendel, Palmer & Tritton, Kalighat, Calcutta, India.

‡ Received by the Secretary, April 25, 1930.

§ *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2653.

from this that for the safety of the bridge, marginal factors of 2⅓ for dead load and 3½ for live load are quite ample?

On the other hand, the writer differs with the Committees in regard to the proposed new impact formula (Article 204*). As will be seen from Fig. 16 the old formula, I_3, would be preferable because it approaches tests results, I_1, more nearly than the new one, I_2. On the other hand, disregarding test results and considering the old formula as correct, the new one should be such as to give higher values since the decreased dead load of the bridge causes the effect of impact to be increased. In other words, the I_1-curve (see Fig. 16) of a bridge designed according to the new specifications would be expected to be somewhat higher. The new formula does give higher values, but only for lengths greater than 137 ft. For lengths less than 80 ft., it gives values that are even less than those determined by tests.

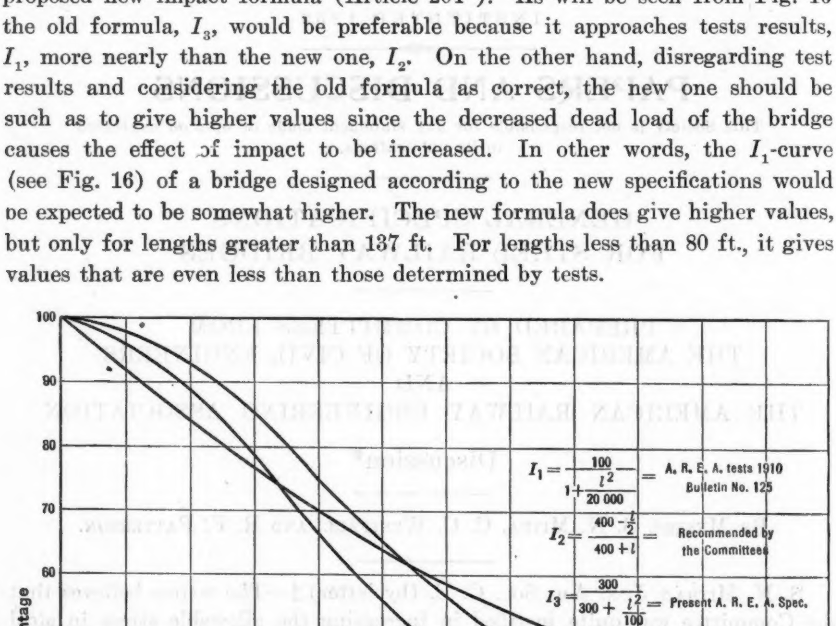

FIG. 16.

The writer, therefore, is of the opinion that until a new I_1-curve is obtained, the old formula would serve the purpose better than that proposed by the Committees. If, by experiment, no appreciable difference is noticed in the I_1-curve (the writer expects the difference to be rather small), then the old formula should be retained.

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2652.

C. C. Westfall,* M. Am. Soc. C. E. (by letter).†—The fact that the following remarks are entirely critical does not mean that the writer has not found much that is commendable in the proposed specifications of the Conference Committee. This discussion is restricted to the subjects of "Loads" and "Stresses," which the writer believes to constitute the important features of a specification and to which a discussion may well be limited. Possibly, the best that can be hoped for from a general specification is that it will be adopted by engineers as a guide for individual specifications; and, in such cases, the sections on "Loads" and "Stresses" will, no doubt, be used in entirety, while each engineer will doubtless incorporate many of his individual ideas in the sections on "Design."

On being presented with a new specification for steel railway bridges, members of the Railway Engineering Profession will, without doubt, first question the necessity or occasion for the new specifications, and, secondly, the improvement over those now in use.

The specifications of the American Railway Engineering Association, as revised August, 1925, represent a great deal of work by the members of that Association and constitute a very satisfactory specification for steel railway bridges. There are doubtless many features of these specifications which are not accepted by all railroads, but they probably come as close to meeting general approval as any single set can. The writer has not heard a demand for new specifications from railroad engineers.

Possibly the first thought of the writer of a new specification for railway bridges is to originate an engine loading which will be an improvement over the Cooper loadings; and, second, to improve the impact formula. This is not strange, as it has long been recognized that the Cooper loadings do not conform to actual engines in use, and knowledge of impact is far from satisfactory.

In the past, while the subject of wheel loads has been given much thought, railway engineers have, in general, agreed that the Cooper loadings meet their requirements better than any others yet proposed and have continued to use them. In the opinion of the writer, the Cooper system of loadings is not as unsatisfactory as many engineers seem to believe, and with the wide variety of heavy locomotives now in use, and coming into use, it is probable that they will actually be as satisfactory, in most cases, as any one combination of axle loads that can be devised.

The writer has prepared curves of moments and shears for the proposed A-64 loading,‡ together with Cooper's E-60, E-65, and E-70, and the heaviest engines in use on the Illinois Central System, for spans of less than 200 ft. On the shear diagram, the curve for the A-64 loading follows closely the Cooper loading, but is not as smooth. The curves for the Cooper loading satisfactorily follow the diagrams for the actual engines. On the moment diagram, the result is very much the same. The Cooper loading approaches closely the actual engines, provides a smoother curve, and for spans between approxi-

* Engr. of Bridges, Ill. Central R. R., Chicago, Ill.

† Received by the Secretary, May 12, 1930.

‡ *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2651.

mately 105 ft. and 200 ft., the A-64 curve lies between that for Cooper's E-65 and E-70 loading.

It is apparent, therefore, that for spans of this length, which constitute the greater percentage of railroad bridges, nothing is to be gained by disposing of the Cooper loading in favor of the one proposed. Another important consideration, although it is not an engineering point, is that the railroad operating officials have become familiar with the Cooper system and a departure from these loadings would create considerable confusion within railroad organizations.

There seems to be no reason for making a change in the A. R. E. A. specifications in the provision for loading spans carrying more than one track. This is a matter of judgment, and it is probable that the A. R. E. A. specification is entirely safe. It is a question whether present knowledge of the subject of impact is sufficient to warrant offering a new formula varying only slightly from the one included in the A. R. E. A. specifications.* As between the two formulas—curves developed for spans of from 25 ft. to 300 ft. cross at a length of about 135 ft. and show a variation of a maximum of 10.7% at 300 ft. It is very doubtful whether the present formula is within this percentage of being correct in its application to all spans. It would seem illogical to change a formula, which has now been in use for several years, for one which is not based on more exact information.

On the Illinois Central Railroad, impact is computed from the formula,

$$I = \frac{\text{live load} \times \text{live load}}{\text{live load} + \text{dead load}}$$

The writer does not see the logic of computing impact on multiple-track bridges on a basis of live load on one track only.

Referring to Section 3 ("Unit Stresses"†), the writer questions the change in column formula. The A. R. E. A. formula is simpler and is probably as well founded as the one proposed. It seems to be a step backward to adopt Article 300,† providing for an increase of 50% in unit stresses for dead load. This would only mean encroaching upon the safety factor, and past experience would certainly not indicate that this would be a proper thing to do. At the present state of knowledge in this field, the Conference Committees cannot do better than to adopt the A. R. E. A. specifications for loadings and unit stresses and, as far as practical results are concerned, the entire specifications.

R. F. PATTERSON,‡ ESQ. (by letter).§—These specifications seem to be satisfactory and the writer has only one suggestion to offer. Under Part II, describing "Materials",|| a specification should be included for copper steel. For "salty" places, the use of this structural material has proved to be sufficiently good practice to warrant its insertion.

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2652.

† *Loc. cit.*, p. 2653.

‡ Care, Hotel Plaza, Camaguey, Cuba.

§ Received by the Secretary, June 2, 1930.

|| *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2667.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

RAINFALL CHARACTERISTICS AND THEIR RELATION TO SOILS AND RUN-OFF

Discussion*

By Messrs. R. W. Powell, Glen N. Cox, and C. R. Pettis.

R. W. Powell,† Assoc. M. Am. Soc. C. E. (by letter).‡—In making this complete compilation of rainfall data the author has rendered a great service to the Civil Engineering Profession, and his discussion of the data is also helpful. The writer wishes merely to comment on two or three of the many phases treated.

The author states§ that "no direct relation is traceable between rainfall and sunspot periodicity", which seems to dismiss too lightly the results of the extensive study that has been put on this subject. The writer would not go as far as such extreme "cyclists" as Alter,‖ Streiff,¶ or Gillette,** but it seems that there must be some truth behind their computations. The studies of men like Huntington†† and Clayton‡‡ seem to establish a scientific basis for a relationship between solar radiation and rainfall.

Some studies on this subject have convinced the writer that the variations in annual rainfall do not take place purely at random but must be regarded as random variations of fairly large amplitude superimposed upon several cyclic

* Discussion of the paper by C. S. Jarvis, M. Am. Soc. C. E., continued from May, 1930, *Proceedings*.

† Asst. Prof. of Mechanics, The Ohio State Univ., Columbus, Ohio.

‡ Received by the Secretary, March 29, 1930.

§ *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 22.

‖ "An Examination by Means of Schuster's Periodograms of Rainfall Data from Long Records in Typical Sections of the World", by Dinsmore Alter, *Monthly Weather Review*, February, 1926, pp. 44-56; also, several other articles in the same periodical.

¶ "Sunspots and Rainfall", by Abraham Streiff, M. Am. Soc. C. E., *Monthly Weather Review*, February, 1927, pp. 69-71, and a former article in the same periodical for July, 1926, pp. 289-296.

** "Five Important Weather and Earthquake Cycles", by H. P. Gillette, M. Am. Soc. C. E., *Water Works and Sewerage*, October, 1929, pp. 429–432, and "A 22.4-Year Cycle in Rainfall and Its Cause", by the same author, in the same periodical for November, 1929, pp. 477-480.

†† "Earth and Sun", by Ellsworth Huntington, and "Climatic Changes", by Huntington and Visher.

‡‡ "World Weather", by H. H. Clayton, MacMillan, 1923, especially the section on "Relation of the Sunspot Period to Rainfall", pp. 309-314; the map on p. 264; and the chapter on "The Sun and the Weather", pp. 215-269.

variations of smaller amplitude. For example, the following equation was deduced for Boston, Mass.:*

$$y = 44.22 + 8.60 \cos \frac{k\,(x - 1767)}{96} + 2.58 \cos \frac{k\,(x - 1773)}{47}$$

$$+ 2.70 \cos \frac{k\,(x - 1798)}{34} + 2.56 \cos \frac{k\,(x - 1811)}{18} + 1.88 \cos \frac{k\,(x - 1810)}{10} \ldots (1)$$

in which, y is inches of rainfall to be expected in any given year; k is 360 to give the angle, in degrees, or 2π to give it, in radians; and x is the date (A. D.) of the year considered.

The 111 years of record from 1818 to 1928, inclusive, fitted this equation with an average deviation of 4.28 in., while the average deviation from the mean was 6.36 in. Equation (1) is not recommended for use in forecasting the rainfall at Boston; it is given simply to illustrate the fact that after the best combination of cycles has been selected to fit any given set of data, the individual deviations will still be large. However, the cycles concealed by these random deviations may still be real, and the writer believes that one of them will usually have a period of about eleven years, and will synchronize with the variations in sunspots. Equation (1) is purely empirical; the denominator of the last fraction could have been changed to 11 or 11.2; some of the coefficients could have been changed slightly, and the equation would have fitted the data nearly as well.

The other point upon which the writer wishes to comment is variability or dispersion of the amounts of annual rainfall at a given station, when no account is taken of the order of occurrence. When the figures for annual rainfall are arranged in the order of magnitude, what is the result? The author has touched upon this question by presenting Fig. 3,† which is a semi-logarithmic plotting of part of the data for four stations; but for the remainder he has indicated the variability only by giving the maximum and minimum of the observed values. This is manifestly a very uncertain method of describing the variability, because it depends on only the two measurements and neglects all the others; and also because it depends quite a little on the length of the period of record. As Fig. 3 shows, if 20 years of record have indicated a certain range between maximum and minimum, it is probable that 200 years of record at the same station will show a range of about twice as much. If the data for all the stations were plotted in this same way the lines for the stations where the annual rainfall is most variable would be steepest and the slope, or some number depending upon it, could be taken as a measure of the variability.

In a paper entitled, "Flow in California Streams"‡ data are given for annual rainfall for about 260 California stations for the 50 water years, 1871-72 to 1920-21 (or for those of them that are known). Rainfall is expressed both in inches and in percentage of the average rainfall for those years. For a supplementary list of 17 stations the data are given in inches only; 41 of the stations in the first group are included in the author's list. To these, the

* See an article by the writer in *Annals of Mathematical Statistics*, May, 1930.

† *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 24.

‡ *Bulletin No. 5*, Dept. of Public Works, Div. of Eng. and Irrig., State of California.

writer added 4 from the supplementary table and for each of these 45, he plotted a graph. Figs. 9 and 10 are the plottings for Stockton, Folsom, and Sterling. These show the method and illustrate one of the less variable stations, one more nearly the average, and the most variable, respectively. The

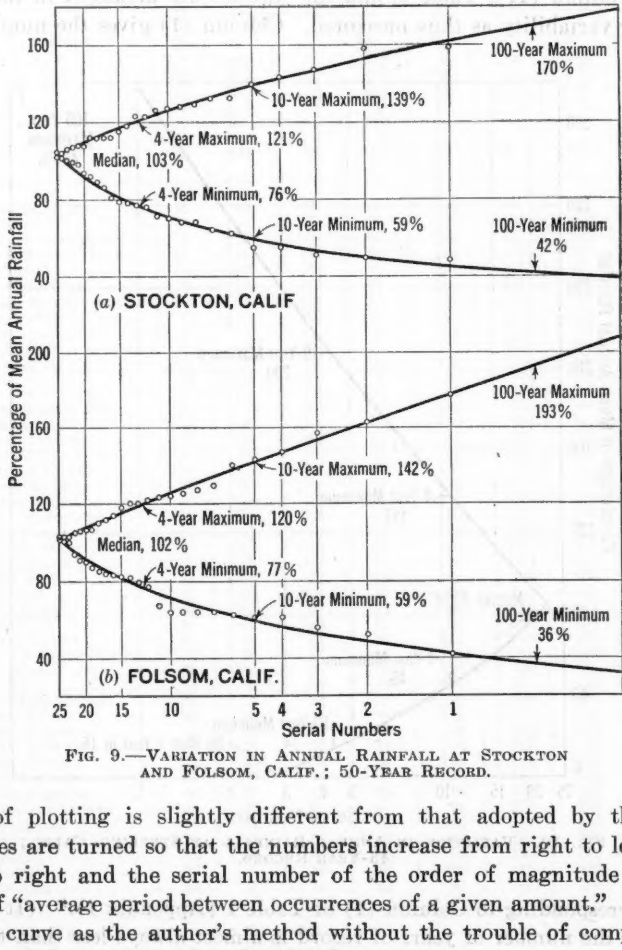

Fig. 9.—Variation in Annual Rainfall at Stockton and Folsom, Calif.; 50-Year Record.

method of plotting is slightly different from that adopted by the author. The curves are turned so that the numbers increase from right to left instead of left to right and the serial number of the order of magnitude is plotted instead of "average period between occurrences of a given amount." This gives the same curve as the author's method without the trouble of computing the average periods or of plotting fractional amounts.

From these curves the expected maxima and minima every 4, 10, and 100 years, and the median values, were read and tabulated in Table 6. The 4-year maxima and minima are the quartile points, such that one-half the values come between them, one-quarter above, and one-quarter below. If the variation were according to the "normal law of error", half the difference between the 4-year maxima and minima would be the "probable error" or the standard deviation multiplied by 0.6745. At first thought, this would seem the best constant by which to measure variability; but it was found that stations with about the same range between 4-year maxima and minima displayed very different ranges

between 100-year maxima and minima. However, as the period of record averages only about 40 years, the 100-year values are somewhat uncertain. It was thought best, therefore, to note the 10-year maxima and minima and to take their difference as a measure of the variability. This difference is tabulated in Column (11), Table 6, and the stations are arranged in the order of increasing variability as thus measured. Column (1) gives the number of the

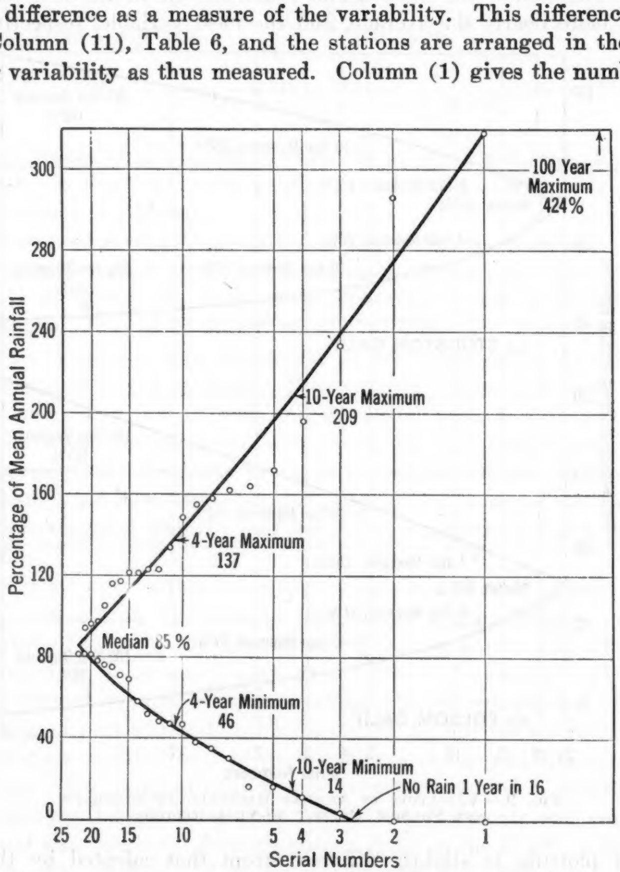

FIG. 10.—VARIATION IN ANNUAL RAINFALL AT STERLING, CALIF.; 45-YEAR RECORD.

station corresponding to Column (1) of Table 1 (Appendix 1).* (It should be noted that the number of years of record is almost always less than that given by the author, because the writer has generally included only data between the years 1871 and 1921). The values given in Table 6 are sufficient to enable one to reproduce the curve, so that the expected variation in any other period of years desired may be determined with an accuracy about as great as is justified by the original data.

The curves given in Figs. 9 and 10 are typical and show that the line resulting from this semi-logarithmic plotting is not generally straight. Some investigation was made of other methods of plotting. Hazen's arithmetic probability

* *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 34 *et seq.*

TABLE 6.—VARIATION IN ANNUAL RAINFALL AT FORTY-FIVE CALIFORNIA STATIONS AS READ FROM SEMI-LOGARITHMIC PLOTTINGS OF THE DATA.

Station No.	Station.	Years of record.	MINIMUM PRECIPITATION. 100 years of record.			Median value.	MAXIMUM PRECIPITATION. 4 years of record.			Variation, for 10 years of record.
			100 years of record.	10 years of record.	4 years of record.		4 years of record.	10 years of record.	100 years of record.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
214	Crescent City	30	62	72	81	94	117	139	183	67
216	Eureka	34	51	70	80	92	115	138	184	68
225	Mount Tamalpais	22	28	64	82	101	118	134	175	70
253	Lick Observatory	40	44	66	80	100	118	137	178	71
260	Auburn	50	44	64	79	101	122	137	156	73
261	Nevada City	50	53	63	77	100	121	136	157	73
262	Lake Spaulding	27	50	63	77	101	123	138	162	75
257	Placerville	50	43	63	77	98	120	138	167	75
256	Tamarack	18	57	65	75	91	113	141	211	76
223	Fort Ross	45	39	64	80	102	123	141	177	77
254	Stockton	50	42	59	76	103	121	139	170	80
250	Fresno	40	46	68	79	96	122	148	200	80
234	Mount Wilson	17	42	67	82	99	129	147	166	80
255	Mokelumne Hill	36	57	64	76	102	125	146	198	82
222	Santa Rosa	33	36	65	81	99	120	147	215	82
215	Fort Gaston	20	56	66	78	99	124	149	200	83
259	Folsom	50	36	59	77	102	120	142	193	83
249	Lemon Cove	21	57	72	83	95	123	157	243	85
217	Weaverville	31	50	58	74	92	119	145	194	87
220	Marysville	50	39	57	73	96	123	146	192	89
226	San Francisco	73	29	60	78	96	119	149	225	89
241	San Bernardino	50	45	53	71	98	122	143	190	90
221	Helen Mine	21	51	60	75	105	137	151	166	91
238	Cuyamaco	33	43	61	75	96	131	152	172	91
219	Red Bluff	44	37	62	78	98	127	153	205	91
252	Merced	49	30	58	76	97	121	149	203	91
229	Monterey	37	37	58	72	94	130	149	212	91
264	Truckee	50	31	58	75	96	124	150	202	92
246	Bakersfield	31	31	63	84	105	130	156	212	93
233	Lowe Observatory	21	20	47	66	95	124	141	161	94
237	Campo	31	36	55	73	99	128	151	163	96
218	Delta	39	40	54	70	101	125	150	214	96
228	Boulder Creek	28	22	57	78	102	126	154	215	97
227	San Jose	47	23	55	74	96	124	153	217	98
258	Sacramento	73	29	52	72	95	121	152	220	100
230	San Luis Obispo	50	29	54	79	92	126	155	205	101
236	San Diego	72	35	51	68	95	123	153	217	102
245	Tehachapi	37	36	53	70	98	129	161	228	108
232	Los Angeles	44	32	51	68	96	127	160	245	109
231	Santa Barbara	50	30	51	67	90	129	162	209	111
235	Riverside	40	37	49	65	96	130	162	228	113
248	Independence	29	28	51	69	101	147	203	342	152
240	Indio	43	10	28	48	90	150	190	217	162
244	Mohave	37	*	19	48	100	146	193	311	174
239	Sterling	43	†	14	46	85	137	209	424	195

* No rain 1 year in 31 years (on the average).

† No rain 1 year in 16 years (on the average).

paper,* gives a line more curved than the semi-logarithmic. Hazen's logarithmic probability paper† is still worse. Goodrich's skew probability paper‡ was also tried and seemed about equal to Hazen's arithmetic paper. Hall's hydraulic probability paper§ gave an almost straight line, as was to be

* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 220.

† *Loc. cit.*, p. 221.

‡ "Straight Line Plotting of Skew Frequency Data," by R. D. Goodrich, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 91 (1927), pp. 1–118.

§ "The Probable Variations in Yearly Run-Off as Determined from a Study of California Streams", by L. Standish Hall, Assoc. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), pp. 191-257; Fig. 4, p. 211.

expected since it was derived from the flow of California streams; but when data for the Northeastern United States and Northwestern Europe were plotted, it did not give as straight a line.

A further study of the slope of the frequency curves was made by using the composite data from twenty of the stations, which gave a total of 1 000 years of record. The stations used, in ascending order of variability, were Auburn, Nevada City, Placerville, Stockton, Folsom, Marysville, San Francisco, San Bernardino, Merced, Truckee, Bakersfield, Sacramento, San Luis Obispo, San Diego, Los Angeles, Santa Barbara, Independence, Indio, Mohave, and Sterling. It must be admitted that this selection is somewhat overweighted with stations of high variability and that the extreme values from a few stations appear very prominently, so that the resulting curve (Fig. 11), obtained by using these 1 000-year data as though they were all obtained at one station, is hardly typical of the State as a whole, but of its more variable portion. In other words, the middle portion of the curve, excluding, say, the upper 50 and lower 50 of the 1 000 values, is fairly typical of the State as a whole, while the "tails" are typical of the most variable stations.

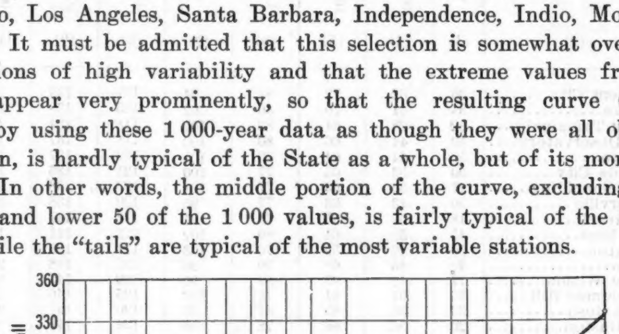

FIG. 11.—VARIATION IN ANNUAL RAINFALL AS ILLUSTRATED BY COMPOSITE DATA FROM TWENTY CALIFORNIA STATIONS, TOTALING 1 000 YEARS OF RECORD.

In Fig. 12, the frequency distribution of the California record is compared with that of two other series of composite rainfall records totaling 1 000 years of record each.* One consists of records from ten stations in the Northeastern United States, as follows (years of record given in parentheses); Boston (110); Lowell, Mass., (99); New Bedford, Mass. (108); Providence, R. I. (93); Albany, N. Y. (103); New York, N. Y. (103); Rochester, N. Y. (93); Philadelphia,

* From data in *Monthly Weather Review*, 1924, p. 485.

Pa. (103); Lebanon, Pa. (82); and Baltimore, Md. (106). The other consists of records from eight stations in Northeastern Europe, as follows: Paris, France (186); Lund, Sweden (160); Kendal, England (135); Edinburgh, Scotland (120); Greenwich, England (109); Tilsit, Germany (105); Copenhagen, Denmark (100); and Brussels, Belgium (85). The points for Northeastern United States and Northwestern Europe agreed so nearly that one curve was drawn to represent them both.

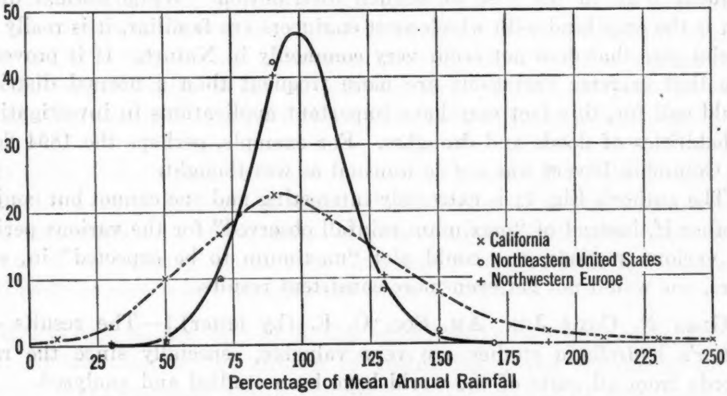

FIG. 12.—COMPARISON OF RAINFALL FREQUENCY RECORDS.

These data were also investigated by the methods given by Fry.* Using his notation, the data gave the values listed in Table 7.

TABLE 7.—COMPARISON OF RAINFALL FREQUENCY RECORDS.

Symbol.	Definition.	California stations.	Northeastern United States and Northwestern Europe.
σ	Standard deviation, in percentage of annual rainfall.	40.3	17.1
$\sqrt{\beta_1}$	Asymmetry, or skewness	0.940	0.226
β_2	Flatness	5.69	3.64
J	Type criterion	− 2.73	− 1.13

These distributions (Table 7), therefore can be represented by Pearson's Type IV, or by a Gram-Charlier series, or perhaps by some form not considered by Fry, such as that of Krichevsky.

While no great importance can be attached to the values for California because the data are not really homogeneous enough to be combined, it is interesting to note that the California streams investigated by Hall† gave a "coefficient of variation" of 0.48, while the rainfall gives 0.403 (since the mean is 100% and "coefficient of variation" is defined as standard deviation ÷ mean).

* "Probability and Its Engineering Uses," Chapters VIII and IX.

† *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 191.

H. R. Tolley states* that several hundred years of data are necessary before the fourth moment (which is what determines the "flatness") can be determined accurately. This is probably true, but the writer believes that it is allowable to build up composite data from stations having similar characteristics. The writer believes that further study will show that ordinarily rainfall and run-off will be found to have a "flatness" of more than 3 (the value for "normal" distribution) and that extreme variations therefore will be more frequent than in the case of normal distribution. While normal distribution is the only kind with which most engineers are familiar, it is really a very special case that does not occur very commonly in Nature. If it proves to be true that extreme variations are more frequent than a normal distribution would call for, this fact may have important applications in investigating the probabilities of floods and droughts. For example, perhaps the 1894 flood on the Columbia River† was not so unusual as was thought.

The author's Fig. 4‡ is extremely interesting and one cannot but conjecture whether if, instead of "maximum rainfall observed" for the various periods at the various stations, one could plot "maximum to be expected" in, say, 10 years, one would not get even more consistent results.

GLEN N. COX,§ JUN. AM. SOC. C. E. (by letter).‖—The results of the author's hydrologic studies are very valuable, especially since the rainfall records from all parts of the world have been studied and analyzed.

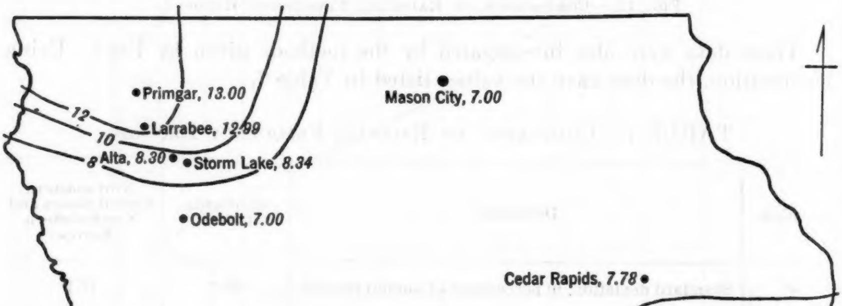

FIG. 13.—MAXIMUM 24-HOUR RAINFALL IN THE STATE OF IOWA, IN INCHES.

* "Frequency Curves of Climatic Phenomena," *Monthly Weather Review*, 1916, pp. 634-642.

† *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 223.

‡ *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 25.

§ Assoc. Prof., Dept. of Civ. Eng., Louisiana State Univ., Baton Rouge, La.

‖ Received by the Secretary, April 8, 1930.

There is one consideration that should be kept in mind. With such an exhaustive study as this, the final results may not represent the facts in every detail. For example, in Fig. 13, the maximum rainfall of Iowa is shown as varying from more than 4 in. in the southeastern part to more than 7 in. in the northeastern and southwestern parts.

The values in Fig. 13 were taken from the Weather Bureau records published by the U. S. Department of Agriculture. These results show that an erroneous impression might be obtained from Fig. 5* as presented by the author. Bonaparte, Iowa, is in the southeastern part of the State, practically on the 5-in. isohyetal line in Fig. 5; yet the records show a measured rainfall of 12.10 in. from this station. Larrabee, with its 12.99 in. of measured rainfall, would be practically on a 6-in. isohyetal line. In Fig. 13 note the widespread distribution of the stations which have had measured rainfalls in excess of 9 in.

It is quite evident that such a study as that presented by the author can only furnish general indications and that the records must be consulted for detailed information on the locality in question. It would sometimes prove disastrous if engineering works were designed to handle such low values of rainfall as those shown in Fig. 5 for the State of Iowa.

C. R. Pettis,† M. Am. Soc. C. E. (by letter).‡—Table 1 (Appendix I)§ is a distinct contribution to the subject of meteorology, and should be in the library of every engineer who is interested in the subject of river flood flow It contains a great deal of information in convenient form, and from it an engineer can obtain some idea about the rainfall characteristics of nearly any place in the world. The author has performed a difficult task, and he is to be congratulated on the result.

The comments made in this discussion are based on the relation of rainfall to river flood flow, and with particular reference to maximum precipitation and maximum floods. In most flood control work hydrologists are mainly interested in the maximum quantity of rain to be expected in one storm. The book entitled, "Storm Rainfall of Eastern United States",‖ is an excellent study of storms in relation to their possible flood-producing qualities. In this report storm precipitation is considered for various periods of time, from one day to six days. In Table 1, the author gives precipitation data for one hour, one day, and one month; of these three periods the one-day precipitation gives the best indication of what may be expected in one storm.

In Table 1 (Appendix I), the flood-control engineer may use the 24-hour precipitation as an index, but he should keep in mind that the actual rainfall in one storm will generally be greater than the precipitation in 24 hours. Ordinarily, the one-storm rainfall will be greater than the one-day rainfall by about 50 per cent.

The 24-hour precipitation is the most valuable part of Table 1 for a flood-control engineer. The 24-hour data are missing for a large number of stations.

* *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 26.

† Lt. Col., Corps of Engrs., U. S. A., Fort Logan, Colo.

‡ Received by the Secretary, March 31, 1930.

§ *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, pp. 32 *et seq.*

‖ Technical Rept., Pt. V, Miami Conservancy Dist.

For example, in Colorado, the values are given for only three out of fifteen stations. (See Items 281, 289, and 293.)

Table 8 gives the maximum observed precipitation, "day of 24 hours," for certain stations in Colorado, obtained from the U. S. Weather Bureau, and they may be used to fill in Table 1.

TABLE 8.—OBSERVED RAINFALL FOR VARIOUS PERIODS, IN COLORADO.

Station No.	Station.	Maximum observed precipitation (day of 24 hours).	Station No.	Station.	Maximum observed precipitation (day of 24 hours).
280	Steamboat Springs..	2.7	287	Longs Peak.........	4.8
282	Durango	2.7	288	Greeley..............	3.4
283	Wagon Wheel Gap..	2.6	289	Denver...............	6.5
284	Gunnison	1.6	291	Lake Moraine.......	5.5
285	Marshall Pass.......	1.5	292	Colorado Springs...	4.3
286	Leadville............	4.0	294	Las Animas.........	3.4

The author's discussion is interesting and instructive. It would appear that he might have made some more specific deductions after so much study; but the information is given in convenient form for a reader to make his own deductions.

The author has one paragraph entitled "Determinate Limits",* in which he indicates that "it has long been recognized that there may be somewhat definite limits of rainfall intensity, frequency, duration, and variation applicable to given localities", and he adds that "an appreciation of the limited scope of human knowledge compared with the broad fields awaiting exploration deters the expression of opinions among those most competent to speak." In various other paragraphs the author evidently has the idea of "Determinate Limits" in mind. The writer wishes to express the opinion that for both practical and theoretical reasons it is much wiser to assume that there is no such thing as a determinate limit of rainfall intensity for any given locality. An effort will be made to reduce the question of determinate limits to concrete terms, and then some of the facts will be given upon which the writer has based his opinion.

According to a formula† introduced by the writer, the flood discharge of a stream varies directly with the precipitation in the storm that produces it, when the conditions are favorable for a heavy run-off, and when seepage can be practically neglected. In the United States, east of the Mississippi River seepage can be generally neglected if only the largest floods are being considered. In the arid West seepage is more of a factor, but even there, in the larger storms it seems to be almost negligible. In any case the relation between precipitation and run-off, or discharge, is such that if there is a determinate limit for the one there is a determinate limit for the other; and, similarly, if there is no determinate limit.

* *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 7.

† Meyer's "Elements of Hydrology," Second Edition, p. 378.

The relation between precipitation and run-off is further confirmed by a study* made by F. G. Switzer, Assoc. M. Am. Soc. C. E., and Mr. H. G. Miller.

The Foster method of applying the Pearsonian theory of probabilities, as described† by H. Alden Foster, Assoc. M. Am. Soc. C. E., has been adopted by a number of engineers. In using the Foster method for floods, the hydrologist must decide upon the type of curve to use for any particular set of observations. For all practical purposes the discussion may be limited to Type I and Type III. Type I has a definite upper limit, or determinate limit. Type III has no definite upper limit; or, in other words, the Type III probability curve is asymptotic to the X-axis, as is the case with the probability curve in the theory of least squares.

The question of determinate limits is thus closely related to the question of the relative merits of Type I and Type III curves.

No engineer can design flood-control works with the meager data generally found in the United States, without basing his design on probabilities. The writer believes that the Foster method will come into wider use by engineers when its possibilities and limitations become better understood.

The question of determinate limits, whether decided one way or the other, will not have an unreasonable effect on engineering design. No matter what the long-time probabilities may be, there is always a determinate economic limit to the amount of money that may wisely be spent on an engineering project. It would not be wise to build levees to protect against a probable 1 000 000-year flood, any more than it would be wise to build houses in Kansas to protect against the worst tornado to be expected once in 1 000 000 years. It is essential that the engineer should have a definite conception of the degree of safety that he is attaining, and that there should be full value, for money spent, in safety, or other desirable features.

To illustrate the practical difference between determinate limits, and no determinate limits, attention is invited to the following figures which are taken from computations made for Bear River, Collinston, Utah:

	Cubic feet per second.
Type I Curve; Determinate Limit:	
10-year flood	10 000
100-year flood	12 000
10 000-year flood	15 000
Type III Curve; No Determinate Limit:	
10-year flood	10 000
100-year flood	14 000
10 000-year flood	22 000

Attention is invited to the fact that for the Type I curve the 10 000-year flood is only 50% greater than the 10-year flood. This is a small spread, and would indicate that the river is very regular in its habits.

This illustration is given in order to help visualize the problem. If it is assumed that there is a determinate limit, this would imply a greater degree

* "Floods," by F. G. Switzer, Assoc. M. Am. Soc. C. E., and H. G. Miller, *Bulletin*, Eng. Experiment Station, Cornell Univ., Ithaca, N. Y.

† "Theoretical Frequency Curves and Their Application to Engineering Problems," by H. Alden Foster, Assoc. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 142.

of regularity in the flood habits of rivers than seems reasonable to the writer. On the other hand, even if there is no determinate limit to rainfall or floods, there will still be a definite limit for economic engineering design.

If the Type III curve is adopted for the Bear River, it would still appear to be a reasonably steady stream, and quite satisfactory from the standpoint of engineering design. The writer has had occasion to use the Foster method with a number of streams of different sizes, and it is thought that the Bear River may be taken as fairly typical of probability characteristics.

The following is taken from the writer's notebook: The maximum flood on the Susquehanna River, at Williamsport, Pa., between 1895 and 1914, was 141 000 cu. ft. per sec. The 1889 flood was 350 000 cu. ft. per sec. Comparing this with the Bear River it could be assumed tentatively that the 1889 flood was about in the 10 000-year class.

The Juniata River, at Huntingdon, Pa., had a maximum discharge between 1896 and 1914 of 37 800 cu. ft. per sec. The 1889 flood was 106 000 cu. ft. per sec., which would seem to be considerably greater than the 10 000-year flood. The writer is of the opinion that neither one of the two 1889 floods was as unusual as indicated.

Another example: The maximum flood on the Arkansas River at Pueblo, Colo., between 1895 and 1920 was 17 500 cu. ft. per sec. The maximum flood in 1894 was 39 100 cu. ft. per sec. The maximum flood in 1921 was 103 000 cu. ft. per sec. This would indicate that the Arkansas River in Colorado is not in the same class with the Bear River in the neighboring State of Utah. In 1920, the 1894 flood might have been considered near the limit. Then, how should the 1921 flood be classed?

Consider now some of the author's figures for rainfall. In Fig. 5* is represented the maximum 24-hour rainfall in the United States. In the Eastern and Central United States the isohyetals seem to be fairly well smoothed out so as to represent average conditions, except for one place in Southeastern Georgia, and another place in Texas. These two "humps" are undoubtedly due to two particular storms, and they not only stand out above anything in the vicinity, but above any other place in the United States. If the isohyetals in Texas surrounding the high spot are typical, then the one storm in Texas set a high determinate limit for one particular part of Texas. The 23-in. isohyetal is surrounded on three sides by the 8-in. isohyetal.

In Colorado, on the other hand, the isohyetals do not indicate anything out of the ordinary. In Table 1 (Appendix I), the author gives the maximum 1-day rainfall at Pueblo as 2.9 in., based on a 35-year record. According to his remarks on "Climatic Stability",† a 35-year record may be considered to be fairly good. Nevertheless, there is evidence‡ that in 1921 there was a fall of about 12 in. of rain in 24 hours, within 6 miles of Pueblo. Furthermore, there is reason to suspect that in the same storm, in one or two small areas, there was an intensity of rain that might be compared to the Texas storm, which set the record for the United States.

* *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 26.
† *Loc. cit.*, p. 6.
‡ *Water Supply Paper 487*, U. S. Geological Survey, p. 13.

Fig. 14 is based on the Weather Bureau records of 194 stations, in which the length of records varied from 5 years to 58 years. Isohyetals are based on the highest values of each group. The map is offered as a supplement to Fig. 5, and as illustrating the need for more detailed studies, especially where the topography is irregular.

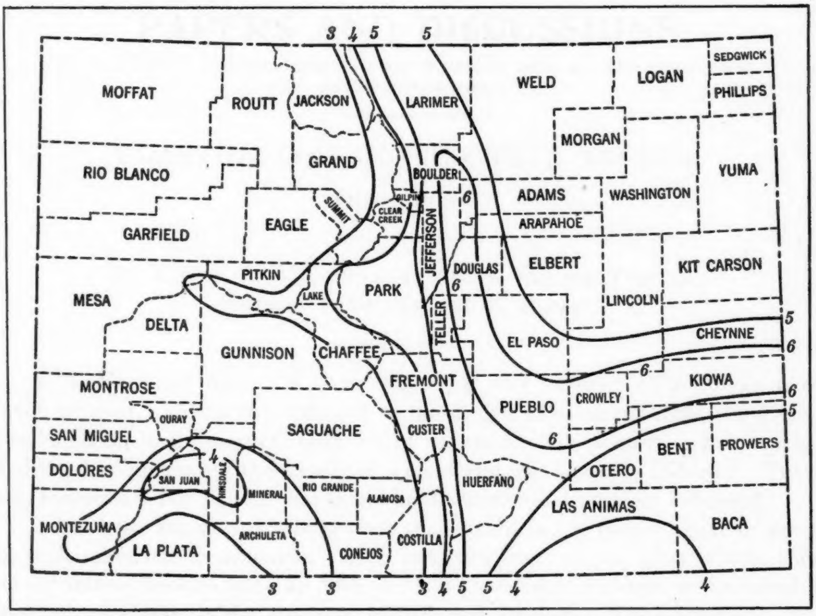

FIG. 14.—MAXIMUM 1-DAY RAINFALL, FOR COLORADO, IN INCHES.

"Cloudbursts" should be considered in any detailed study of Colorado weather conditions. It happens that none of the Colorado weather stations has experienced the most severe type of cloudburst, and this should be considered in using the isohyetal map. Largely on account of the topography, storm rainfall in Colorado is much more irregularly distributed than in the Eastern States.

There is need for further research and study along the lines of this discussion. In Mr. Foster's paper previously referred to, he states that "if in doubt as to which type curve to adopt, Type III may be selected as giving results on the safe side". The writer concurs. He would not adopt a Type I curve, "Determinate Limit," for any river with uncontrolled flow, unless he had sufficient evidence to controvert most of his previous experience along this line.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

PLASTIC FLOW IN CONCRETE ARCHES

Discussion*

BY MESSRS. A. A. EREMIN, J. MELAN, FREDRIK VOGT AND HERBERT J. GILKEY, AND E. PROBST.

A. A. EREMIN,† ASSOC. M. AM. SOC. C. E. (by letter).‡—This paper is of much interest. It is a step forward in the design of concrete arches by giving consideration to the natural behavior of concrete. The subject is well treated and the development of the formulas is clear and can be easily followed. There are, however, a few phases that could be discussed and possibly extended.

It has been known for a long time that strain is not proportional to stress in such materials as concrete, stone, and cast iron; or, in other words, they do not follow Hooke's law. In 1896 C. Bach§ published a paper on the variable modulus of elasticity for concrete at different stresses. Many attempts have been made since that time to develop the formulas for structural members with variable moduli of elasticity. Even for a beam with simple loading, the time required for the calculation of the stresses by means of these formulas, was prohibitive. Bach‖ proved that, in the formulas containing the variable moduli of elasticity, the position of the neutral axis varies with the stresses and also with the change in the external concentrated loads and bending moments.

Max Ritter¶ analyzed an arch by the ordinary elastic method with the constant modulus of elasticity and also by the method with the variable modulus of elasticity, using the exponential expression for strain similar to that used by the author. In conclusion, he stated that for the arch where no tension occurs there is no practical difference in the stress obtained whether a constant or a variable modulus of elasticity was used. This supports the author's con-

* Discussion of the paper by Lorenz G. Straub, Jun. Am. Soc. C. E., continued from May, 1930, *Proceedings*.

† Asst. Designing Engr., Bridge Dept., State Highway Comm., Sacramento, Calif.

‡ Received by the Secretary, April 3, 1930.

§ "Hütte," des Ingenieurs Taschenbuch, 1, 1920, p. 500.

‖ "Elasticität und Festigkeit," 1911, p. 236.

¶ *Schweizerische Bauzeitung*, January, 1907, p. 25.

clusion* that "for smaller structures the usual procedure for stress analysis is probably satisfactory". It is interesting to note that Ritter in his study of arch stresses limited his conclusion to the arches in which no tension occurs because like Mr. Straub in the formulas for variable modulus of elasticity, he assumed that the exponential expression for strain of concrete in tension is the same as in compression. This assumption greatly simplifies the calculations although, at the same time, the formulas developed cannot be used for pure bending stresses without appreciable error.

The actual stress diagram in the plain concrete in pure bending will be shown by the curved line, *A B*, Fig. 31.† When a constant modulus of elasticity is used the stress-strain line will be the straight line, *A'B'*, and the neutral axis will pass through the center of gravity of the section (Fig. 31).

The stress-strain diagram for concrete varies with the grade of cement, aggregate, water ratio of mix, the method of pouring of green concrete, and the age of the concrete. Various curves for the stress-strain diagram due to these factors will be similar, but will show a marked variation in tension and compression, Line *AB* (Fig. 31).

Some of the European specifications adopted the straight-line variation of the moduli of elasticity of concrete, as shown by the lines, *A''O* and *OB''* (Fig. 31). This is somewhat nearer the actual stress-strain diagram and the constant ratio of the moduli of elasticity in compression and tension, simplifying the formulas for stresses. In the general study of elasticity and plastic flow of concrete under pure bending stresses where numerical values are not essential, the assumption could be made that the modulus of elasticity of concrete in tension varies at the same rate as in compression, since this permits considerable reduction in the formulas.

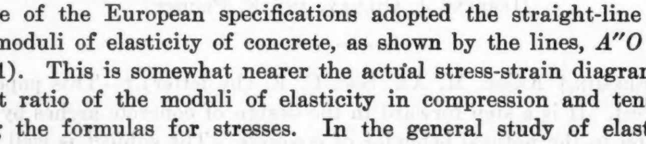

FIG. 31.

In the author's "First Conclusion"‡ he states that:

"It is incorrect to assume that in general plastic flow in statically indeterminate concrete structures 'causes a redistribution of bending moments tending to relieve the overstressed parts of the structures".

The overstressed part of a statically indeterminate structure is distinguished by excessive deformation which will affect other parts of the structure

* *Proceedings,* Am. Soc. C. E., January, 1930, Papers and Discussions, p. 114.

† "Strength of Materials," by Prof. S. P. Timoshenko, Russian Edition, 1916, p. 234.

‡ *Proceedings,* Am. Soc. C. E., January, 1930, Papers and Discussions, p. 113.

similar to some external disturbances which were discussed by the author in his "Second Conclusion".*

Take, for instance, a beam with one end fixed and the other end simply supported, as shown by the author in Fig. 21.† The overstressed part in the middle of the span will produce excessive deflection of the beam, and, at the same time, the reversed bending moment at the fixed end will be increased. Due to plastic flow, a redistribution of the stresses will occur, similar to that caused by the settlement of a support.

Kern analyzed a concrete arch with fixed ends and found that the resultant stresses due to dead load, rib-shortening, and temperature changes, computed by his method‡ with variable moduli of elasticity, are smaller than those computed by the ordinary method with constant moduli of elasticity. He thus proved the author's "Conclusion"* that temperature "stresses computed by the ordinary elastic theory are * * * on the side of safety".

In conclusion, it may be stated that Mr. Straub has made a valuable contribution to engineering literature, and that further theoretical and experimental study of elasticity and plastic flow of concrete arches is highly desirable.

DR. ING. J. MELAN§ (by letter).‖—In applying the elastic theory based on Hooke's law to the design of concrete structures, it is recognized that only approximate values of deformations and stresses are obtained. The assumed proportionality between deformation, or shortening, and stress actually does not exist in materials used for masonry structures: First, because the ratio of stress to strain is not a straight-line relation; that is, the modulus of elasticity is not constant for all stress intervals; and, second, because permanent plastic deformations occur which are dependent on the age of the concrete and the period of loading. The inaccuracy of computation is less, however, the longer the concrete is allowed to set before it is loaded.

It is not easy to set up mathematical laws accounting for the stress-deformation relations of concrete. Hence, the treatment given this subject by Mr. Straub who proposes such relations, is of particular interest.

As a basis for elastic deformation, he uses the Bach-Schüle law,¶ $\epsilon = k\,\sigma^m$, and for plastic deformation he develops another equation** of the exponential type as follows, $\rho = K\,\sigma^p\,t^q$, in which, the deformation is a function of the stress, σ, and the time, t. According to available data for time-yield experiments, although these are few, this assumption seems to define the law of plastic deformation for concrete quite well. The coefficient and exponents are dependent on the composition of the concrete and its age.

The theory derived for fixed-ended continuous arches based on the foregoing equations is mathematically sound, assuming that plane sections remain plane, and if the arch undergoes only compressive stresses, and no tensile

* *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 113.

† *Loc. cit.*, p. 85.

‡ *Beton u. Eisen*, 1930, p. 29.

§ Tech. Univ. of Prag, Prag, Czecho-Slovakia.

‖ Received by the Secretary, April 7, 1930.

¶ *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 60.

** *Loc. cit.*, p. 70.

stresses or only small ones, as a result of the loading and deformation. With these stipulations, the conclusions drawn by the author are also correct. However, the conclusion that the early removal of centering from a reinforced concrete arch is advantageous to the structure, cannot be accepted without reservation. It has not been demonstrated in practice, and conflicts with the rules of construction maintained to the present time, deviation from which may prove to be a dangerous venture.

In any case, Mr. Straub's treatment provides a valuable and meritorious supplement to, and a more thorough analysis of, the theory of concrete arches.

FREDRIK VOGT,* ASSOC. M. AM. SOC. C. E., and HERBERT J. GILKEY,† M. AM. SOC. C. E. (by letter).‡—The author concludes his paper with the words:§

"Much more experimental work, especially of a quantitative nature, is desirable and necessary in order to establish definitely the laws of plastic flow for concrete of various mixes and ages."

This is doubtless true, and for that reason the development of an imposing mathematical analysis upon assumptions of questionable validity appears to be premature at this time. Important problems regarding the qualitative as well as the quantitative nature of flow are still unsolved.

To what extent, for example, is flow plastic and to what extent is it elastic? Most structural materials have a distinct elastic recovery when a stress of any intensity is released. Professor F. B. Seely‖ alludes to the energy of this elastic recoverance as hyper-elastic resilience, as distinguished from elastic resilience (to which the term, resilience, is generally applied). Concrete has this same power of partial recovery within all ranges of stress. That it exists to an appreciable extent, there is no question.

For dead-load stresses the elastic recovery is of no importance, since there will be no release of load; but it is important in connection with stresses due to temperature changes in statically indeterminate structures since these changes have an annual or a seasonal period.

From observations too limited to generalize other than to state that appreciable recovery is a reality, the writers have found that concrete, after unloading, will immediately recover an amount about equal to the initial elastic deformation. In addition to this first recovery there seems to be a slow additional recovery of part of the flow.

Another unsettled problem is the connection between volumetric changes (shrinkage and swelling) due to moisture changes and the flow due to load. The author states¶ that:

"Shrinkage is distinguished from plastic flow in that it is a change in dimensions independent of external forces acting on the specimen."

Let the change in length of an unloaded specimen at constant temperature be termed shrinkage (or swelling). Then, the difference between this shrinkage

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† Associate Prof., Civ. Eng., Univ. of Colorado, Boulder, Colo.

‡ Received by the Secretary, April 14, 1930.

§ *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 114.

‖ "Resistance of Materials," pp. 15 and 361.

¶ *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 90.

and the change in length of a specimen exposed to a continuous load is termed flow. The writers know of no evidence that the flow determined in this way is independent of the shrinkage, or, to go a step further, independent of moisture changes and changes in the fine structure of the concrete, such as re-crystallization, etc. On the contrary, there is some reason for suspecting that there may be a connection between the two phenomena. It is a striking fact that concrete specimens under continuous compressive loading, stored in air, flow several times as much as if stored under water.* The writers have described, elsewhere,† the result of their own experimental work on the subject. In air, the shrinkage and the compression due to the load co-operate in reducing the length, while in water the swelling under water counteracts the compression due to load. The influence of the load taken separately, that is, the flow, is several times as great in the case of co-operation with shrinkage as it is when counteracted by swelling, even after the direct shrinkage and swelling (as determined from unloaded specimens) have been deducted.

While the difference is due in part no doubt to the difference in physical condition, it seems possible and probable that the loaded or unloaded state in combination with the dry or wet condition may have a distinctive influence upon such internal phenomena as re-crystallization. Attempts have been made to supplement the available information by duplicate tests under continuous tensile loading in both air and water. The tensile flow tests have not been satisfactory on account of the low range of strain that is available in tension. It is to be noted that, with moduli of elasticity nearly equal in tension and compression and the ultimate tensile strength about one-tenth the ultimate compressive strength, the tensile deformation available is only about one-tenth of that for compression.

Bending of beams under continuous loading was easier to handle and gave results that were in all respects similar to those obtained in compression. The beams represent a mixed condition with both tension and compression present, and the results are, therefore, more difficult to interpret.

If the flow and shrinkage or swelling are interrelated and interdependent, the fact will have an important bearing upon the stresses due to flow. This is particularly true in bending and where temperature alternates the stresses. It also has a bearing on the question of recovery.

The author has based his deductions on a formula for the total deformation,‡

$$\varepsilon_{total} = \varepsilon + \rho = k\,\sigma^m + K\,\sigma^p\,t^q \;\ldots\ldots\ldots\ldots\ldots (116)$$

as an extension of the Bach-Schüle exponential formula for the initial deformation given by the first term.

The Bach-Schüle formula gives a fair approximation to the stress-strain diagram for high stresses, but it is well known that it is not valid for low stresses. For example, with σ approximating zero, the exponential formula gives moduli of elasticity, E, approaching infinitely large values if m is greater

* See the experiments made by R. E. Davis, M. Am. Soc. C. E., Report on Arch Dam Investigation, *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 211.

† *Proceedings*, Am. Soc. for Testing Materials, Vol. 29, Pt. II, 1929, p. 706.

‡ *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, pp. 59, 70.

than unity, as it is for concrete.* This result is not in agreement with experience and greatly limits the formula in its application. Similar limitations are probably applicable to the second term. It is more important that the exponential term, t^q, with q positive, shall give an ever-increasing flow which approaches no limit or stopping point. The factor, t^q, reaches infinitely high values with increasing age. As a result, an arch bridge, according to the formula, would deflect until it was flattened to the point of failure. Obviously this does not happen, and the law must be different even if the formula gives a fair approximation during the first few weeks or months. In Fig. 14† the check with some experimental results is shown for a period of less than five months, but this does not justify the use of the formula for a period of more than nine years, as was done in Fig. 24.‡ Nor has the author limited his analysis to nine years; he has extended it until the time interval becomes infinite as regards the point of contraflexure for the beam with which he deals. In other words, he has extended it until the deflection, according to his formula, also becomes infinite.

The author's "Second Conclusion"§ states:

"If there is an external disturbance * * *, such as the settlement of a support, the plastic flow causes a gradual redistribution of stresses such that the structure approaches (but never reaches) its original state of equilibrium (no settlement of support)".

This conclusion results from assuming a flow which will cause the modulus of elasticity to approach zero as time goes on. With a secant modulus,

$$E = \frac{\sigma}{\varepsilon} = \frac{\sigma}{k\,\sigma^m + K\,\sigma^p\,t^q} = \frac{1}{k\,\sigma^{m-1} + K\,\sigma^{p-1}\,t^q} \quad \text{........(117)}$$

and, with q positive, E approaches zero as t becomes infinitely great. Thus, the deformation due to a given load increases indefinitely with time, and the effect of initial external disturbances becomes smaller and smaller in comparison with the increasing strains from the regular or primary loading. Finally, they become relatively so small that their influence may be considered to be negligible.

Conclusion 2 can only be reached on the basis of the assumed formula for flow. If the flow does not actually increase indefinitely, but if instead it approaches a limiting value, E will remain a finite value, and the result will be entirely different. If, for example, the flow increases the deformation to three times the initial value, then the influence of the settlement of a support on the stresses will be reduced to one-third of what it was at first, but not to zero since the deformation due to the settlement can be eliminated by one-third of the initial force. Compared with this major result the influence of small deviations from the straight stress-strain diagram becomes unimportant.

As stated by the author,§ the temperature changes and shrinkage are among those external disturbances the influence of which is reduced by the flow.

* Compare, for instance, the German Handbook, "Hütte," Vol. I, Twenty-fifth Edition, p. 541.

† *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 72.

‡ *Loc. cit.*, p. 88.

§ *Loc. cit.*, p. 113.

However, this is only true for slow temperature changes and not for rapid or moderately rapid changes. Of course, annual fluctuations cannot receive the benefit of the flow during more than a few months since their action is reversed.

That the flow does decrease with time is shown quite conclusively by the Davis and Troxell experiments.* These tests show that the flow is asymptotic and that it follows the exponential formula expressed by the author for only a short time. This result is also supported by the experiments of the writers, of which previous mention was made.

The experiments by Professor Davis, already mentioned, indicate that the tendency of flow is to straighten out or flatten the stress-strain diagram where the initial deformation shows a modulus of elasticity, E, decreasing with increasing stress, or where the exponent, m, is greater than unity in the Bach-Schüle formula. In Table 28† of the report of the Committee of Engineering Foundation on Arch Dam Investigation, Column (10) gives the ratio of flow to initial elastic deformation. Within each group the ratio is, generally speaking, lower for high stresses than for low ones and the original curving of the stress-strain diagram (decreasing E with increasing stress) is counteracted to some extent by the flow. The effect of flow is to reduce to a smaller, but more constant, value of E.* Experiments made by the writers support these findings. If they are correct, the conception of stresses and changes of stresses due to flow and the effect of flow on the stresses can be stated rather simply: If the flow gives a total deformation approximately proportional to the stress and greater than the initial deformation (that is, if E is lowered by the flow and the stress-strain diagram is straightened out), then:

(*a*) The errors in the usual computations caused by the fact that the stress-strain diagram is not straight, as usually assumed, are reduced by the flow. Particularly in compression, such as in arched bridges, the original error is comparatively small since the stress-strain diagram is fairly straight for compressive working stresses, and the error is on the side of safety.

(*b*) Stresses due to slow temperature changes and shrinkage are reduced due to the flow. They can be computed approximately by assuming a constant value of the modulus of elasticity, but this modulus must include the initial deformation and the flow due to temperature forces; that is, it is lower than the modulus for short-time tests. The same applies to stresses due to slow settlement of supports. The stresses due to load only are not dependent upon the modulus of elasticity if the stress-strain diagram is straight.

With respect to Conclusion 4‡ it should be mentioned that the straightening of the stress-strain diagram gives the same action as when $m > p$ in the author's notation and this will produce, as he has stated, an effect just opposite to that which he has found. This will also make the author's Conclusion 3‡ rather doubtful.

As the discussion must imply, the writers consider the paper to be misleading in its present form. Nevertheless, they do feel that the concept is excel-

* *Proceedings*, Am. Soc. for Testing Materials, Vol. 29, Pt. II, 1929, pp. 695–699.

† *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 213.

‡ *Loc. cit.*, January, 1930, Papers and Discussions, p. 113.

lent. If, as stress and flow phenomena relating to concrete become better understood, a similar analysis based upon a sounder premise could be made, there is no question but that it would be a contribution to the subject. Already, much additional information has been contributed* toward the fuller and better understanding that should be the basis for such an undertaking.

DR. ING. E. PROBST† (by letter).‡—The very interesting theoretical investigations by Mr. Straub have as their purpose the development of a method of computing stresses in concrete arches which conform to the actual conditions more closely than those assumed in applying the usual elastic theory based on Hooke's law.

It has long been recognized that concrete does not possess a linear relation between longitudinal deformation and stress. Furthermore, it has been known that, for concrete, plastic deformation occurs which is a function of both stress and time. Mr. Straub evaluates these relations from experimental data compiled by various experimenters and expresses the elastic deformation by the exponential formula, Equation (9).§ He expresses the plastic deformation as a function of two independent variables, the stress and the time, in the form,

$$\rho = K\,\sigma^{p}\,t^{q} \qquad (118)$$

The exponents m, p, and q, in these equations, are dependent upon the age as well as other properties of the concrete. Mr. Straub recognizes that, in assuming these values, considerable uncertainty exists, but believes that as a result of systematic investigations it would be possible to pre-determine these parameters with considerable certainty.

Particular care must be exercised in evaluating experimental results because frequently the investigations do not account for all influencing factors satisfactorily, and subsequent computed elimination of such factors as change in length due to shrinkage or temperature variation, may lead to deceptive conclusions regarding the magnitude of longitudinal deformation.

Experience has proved that it is practically impossible to fabricate in the field concrete which has uniform properties. Nevertheless, the static computation for the structure must be completed before erection begins because the dimensions of the structural members are dependent on such computations. Thus, properties of the fabricated concrete must be assumed, which, in the most favorable circumstances, are only mean values. Computed stresses, therefore, are not common with those actually occurring in the completed structures even though careful application is made of the laws which relate, respectively, deformation to stress and stress to time. In addition, there is frequently a distortion of the stress condition as a result of cracks in the concrete which are often unavoidable.

Finally, it is assumed that the stress-strain relations are the same for concrete in tension as in compression, although it is recognized that this is not true in fact. The actual relation between the external and internal forces

* See, for example, *Proceedings*, Am. Soc. for Testing Materials, Vol. 29, Pt. II, 1929, pp. 695–699.

† Prof., Technische Hochschule, Karlsruhe, Germany.

‡ Received by the Secretary, April 25, 1930.

§ *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 60.

of an arch are set up by introducing Equations (9) and (118). It is thereby possible to compare the deviation of computed stresses by the proposed method with those by the customary method of computation.

The application of the elastic and plastic theory proposed by Mr. Straub is to be recommended in the design of structures of unusual dimensions and which justify a more complicated and time-consuming method of computation in order to obtain an actual conception of the stresses.

Under normal circumstances the method of computation based upon the usual theory of elasticity will suffice, although it will give only approximate values because of the approximate relations assumed, such as Hooke's law, the law of superimposition, Maxwell's reciprocal theorem, and others.

In his "Conclusion",* the author correctly points out that the frequent assumption made with regard to statically indeterminate structures, namely, that as a result of "plastic flow" a displacement of moments takes place tending to relieve overstressed regions, is incorrect. Equalization of the stressed condition in a manner resulting in relief of the overstressed parts of the structure, so that the resistance of the material in the parts originally less stressed carries a larger load, can occur only with a structural material having properties such as steel.

In the writer's opinion these very valuable scientific investigations by Mr. Straub serve to provide a more thorough conception of the stress conditions of concrete structures. They supply a valuable supplement and amplification to the theory of arches.

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* *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 113.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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LAMINATED ARCH DAMS WITH FORKED ABUTMENTS

Discussion*

By Fredrik Vogt, Assoc. M. Am. Soc. C. E.

Fredrik Vogt,† Assoc. M. Am. Soc. C. E. (by letter).‡—This interesting paper describes three different features in the design of arch dams, *viz.*, the forked abutments, the overhanging type of "constant-angle arch dam," and the laminated arch elements.

The construction of numerous dams, for instance, Salmon Creek Dam designed by L. R. Jorgensen, M. Am. Soc. C. E., Kerkhoff Dam designed by B. F. Jakobsen, M. Am. Soc. C. E., the Gibson and Deadwood Dams designed by the U. S. Bureau of Reclamation, has proved that single-span arch dams can be used where the sites are about as favorable or less favorable than those shown in Figs. 5§ and 7,‖ and without using high gravity wings. When the arches can be founded directly on the rock without high wings or buttresses, this solution seems to offer many advantages. In some cases, however, the topography of the site may invite the use of forked abutments. The Stewart Mountain Dam, of the Salt River Water Users' Association, is an example of forked buttress and gravity wings. In the analysis of such dams it should not be forgotten that the tangential compression of the buttress will increase the span of the arch and thus will increase the rib-shortening pull and the bending stresses.

There is no doubt that the thrust in the arches is inclined at the abutments as assumed by the author, particularly in sites that require a large ratio of span to height. The writer has previously discussed this matter,¶ and the theoretical analysis given at that time was later checked by model tests. Owing to this inclination low buttresses, one-eighth to one-fifth of the height

* Discussion of the paper by Fred A. Noetzli, M. Am. Soc. C. E., continued from May, 1930, *Proceedings*.

† Docent, Norges Tekniske Höiskole, Trondhjem, Norway.

‡ Received by the Secretary, March 21, 1930.

§ *Proceedings*, Am. Soc. C. E., February, 1930, Papers and Discussions, p. 272.

‖ *Loc. cit.*, p. 281.

¶ *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1272.

of the dam, will not receive much load if used at sites as shown in Figs. 5 and 7. It is a question whether gravity wings should not be limited to such heights, where possible. For instance, in the case of Dam No. 1 shown in Fig. 5, arch elements with the same radius as that used in the design, but with greater central angles, could be used for greater spans, and thus the arch elements could be founded directly on the rock for 75 to 80% of the total height of the dam instead of only 53% as noted in the design, thus reducing the wings considerably. Such a change will give a smaller rib-shortening pull and smaller stresses in the arch elements; it will give practically the same stiffness against deflection* and also practically the same cantilever stresses. In other words, the efficiency of the arch elements will be improved in some respects and, in others, will be virtually unchanged.

However, this possibility of increasing the span of the arch and thus reducing the wings without increasing the radius is due to the fact that rather small central angles of about 90°, have been used at the crest for both Dams Nos. 1 and 2.

As shown by Mr. Jorgensen,† it is preferable to use central angles of about 120°, assuming that the cylinder formula is valid. If analyzed by the now generally accepted elastic theory, including also temperature changes, central angles of 150 to 160° are found to give the minimum of concrete, and, therefore, should be preferred where it is possible to obtain good abutments with such large angles.

In the designs of Dams Nos. 1 and 2 the central angles applied at the base of the buttresses are about 130° and 120°, respectively; that is, they are about as suggested by Mr. Jorgensen. Higher up the span of the arch is unchanged, and it is difficult, therefore, to find any reason for the increase of the radius of the arches and the accompanying reduction to 90° at the crest in the central angles. The latter only reduces the efficiency of the arch elements. The writer has emphasized‡ the fact that this use of a variable radius is in disagreement with the constant-angle arch principle and is a disadvantage. If the intention is merely to obtain overhanging arches, this result could easily be secured by the use of a constant radius and a constant angle for the height of the buttresses. For both designs the direction of the buttresses permit central angles as large as 130 degrees.

The first "constant-angle arch dams" were constructed without appreciable over-hang, but where larger angles can be obtained by this means, an over-hang seems to be well justified. Previous to the Railroad Canyon Dam referred to by the author,§ such overhanging dams have been constructed in Italy (Chiusella Dam, 158 ft. high, completed in 1925‖); and in Mexico (Calles Dam, 208 ft. high, completed in 1928¶). If such over-hang is not to result

* That is, practically the same coefficient, λ, in the deflection formula as shown in *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 564, and Fig. 46, p. 565.

† "The Constant-Angle Arch Dam," by Lars R. Jorgensen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 685.

‡ *Engineering News-Record*, 1928, p. 707.

§ *Proceedings*, Am. Soc. C. E., February, 1930, Papers and Discussions, p. 288.

‖ *Transactions*, First World Power Conference, Lond., 1924, Vol. II, p. 591.

¶ *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1310.

in vertical tension in the cantilevers, pressure grouting must be applied in the radial contraction joints.

The Dordogne River Dam, in France, referred to by the author in Fig. 9,* typifies the spirit of many French engineers. The stresses in the Dordogne River Dam are as much as 800 lb. per sq. in., due to water load only, to which must be added the stresses due to the shrinkage and temperature changes. In this connection reference should be made also to the Selune and Belle Isle en Terre Dams, in France. In all the modern arch dams in France† the thickness of the arches is reduced to the limit—for the two last-mentioned dams to 7 and 7½ in. at 50 and 54 ft. head of water—by using very short spans (about 16 ft.) between the buttresses. The experience with dams in the United States, and in Scandinavia, seems to show that this scrimping in thickness, down to the theoretical limit is not advisable, at least not in cold climates.

In any case, the system applied in the Dordogne River Dam is clear in its action, and the maximum stresses can never be very much greater than the cylinder-stresses. However, the safety of such dams is controlled either by automatic valves, or by hand operations of the dam keeper. If the basins between the arches are not filled as water in the main reservoir is rising (for instance, because the valve openings are clogged by ice), or if for any reason they are emptied before the main reservoir is emptied (for example, by a great leakage or a bursted valve or pipe), the maximum stresses can become more than 4 000 lb. per sq. in., and the dam is in danger of collapsing. Furthermore, if the reservoir is emptied and the basins are not, the dam will fail. The writer has seen an earth dam which failed because it was dependent on a spillway with an automatically moving gate, and because the dam keeper did not maintain that gate in good working order during the cold season. After observing such field practices, he is suspicious of types of dams which depend for safety on factors that may not be proof aginst erroneous operation. Dams ought to be constructed with higher safety factors than most other structures.

The site of the Dordogne River Dam would be excellent for a single-span arch dam, with a span at the crest only 1.8 to 1.9 times the height, which is less than for two-thirds that of American dams higher than 100 ft. The construction cost of a structure of the accepted type scattered over a wide area and with greatly increased excavation and form work, will probably be considerably more than for a single-span arch dam, even if the latter should demand more concrete when designed for the same stresses.

Compared with the French dam, the lamination suggested by the author has the advantage that the action is independent of automatic devices. On the other hand the static action is not so clear. The author states:‡

"Assume an elementary horizontal arch with two arch laminae at a depth of 200 ft. below the crest of a dam. The water pressure at this depth is $62.5 \times 200 = 12\,500$ lb. per sq. ft. The up-stream lamina is assumed to carry one-half this pressure, the other half (6 250 lb. per sq. ft.) being transmitted across the joint to the down-stream lamina. Assuming a coefficient of friction in the asphalted joint of 0.5, the frictional resistance in the joint, therefore,

* *Proceedings*, Am. Soc. C. E., February, 1930, Papers and Discussions, p. 285.

† No more than these three are known.

‡ *Proceedings*, Am. Soc. C. E., February, 1930, Papers and Discussions, p. 280.

would be 0.5 × 6 250 = 3 125 lb. per sq. ft. (22 lb. per sq. in.). This is rather small as compared to the difference in stress in the concrete on opposite sides of the joint, which may be as much as 200 to 300 lb. per sq. in. It is evident, therefore, that the friction in the asphalted joint between the arch laminae is not likely to restrain them sufficiently to alter the stresses materially from those indicated by theory".

It should be remembered that the frictional forces involved must be added from the crown section to the abutment, and that, therefore, the total frictional force on each half of the arch will be 50% of the water load on the same half transmitted to the lower lamina if the coefficient of friction is 0.5. If frictional forces 0.5 times the water load are acting in the joint between the two laminae in the arch shown in Fig. 8* for which the stresses are given in Table 2,† the writer has computed additional stresses at some places as great as 680 lb. per sq. in. for that arch according to the elastic theory. This added to the stresses given in Table 2 would increase the maximum total stresses by 110%, which is far beyond all reasonable limits.

However, the actual result will not be so bad since the friction will redistribute the division of the water load on the two laminae, and the laminated arch therefore will not be stressed more than the assumed homogeneous solid arch, but will act about the same.

Another and better way to get an estimate of the importance of the friction in the joints is to investigate the stresses at the assumed joints as they would be if the arch element was made solid. If the shear stresses in the solid arch, acting along the surface where the joint would be, are smaller than the radial compression acting normal to the same surface, multiplied by the coefficient of friction, no sliding can be expected, and the laminated arch will act as a solid arch. Mr. Jakobsen has analyzed such stresses in thick arches.‡ With the simplifications which can be used for slender arches the writer has computed these stresses for the arch element shown in Fig. 8, and finds at the surface of the suggested joint the stresses due to the water load, as follows:

Shear Stresses:

At the crown section,

$$\tau = 0$$

at the abutment section,

$$\tau = 0.0411\,\sigma_0$$

the average for the total length,

$$\tau = 0.0228\,\sigma_0$$

Radial Stresses Normal to the Surface:

At the crown section,

$$\sigma_r = 0.0452\,\sigma_0$$

at the abutment section,

$$\sigma_r = 0.0753\,\sigma_0$$

the average for the total length,

$$\sigma_r = 0.0533\,\sigma_0$$

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* *Proceedings*, Am. Soc. C. E., February, 1930, Papers and Discussions, p. 282.

† *Loc. cit.*, p. 283.

‡ *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 503; see, particularly, Fig. 17, p. 505.

in which, the values of σ_0 are the stresses according to the cylinder formula. With a coefficient, $f = \frac{\tau}{\sigma_r}$, the friction will prevent all movements and make the arch act as a solid one. The limit of the coefficient of friction is in this way found to be 0.00 at the crown section, $\frac{0.0411}{0.0753} = 0.545$ at the abutment section, and an average for the total length of $\frac{0.0228}{0.0533} = 0.427$.

For very thick arches this limit approaches unity; for instance, for $t = 0.5r$ and $2\phi_1 = 90°$, the limit is 1.0 at the abutment and about 0.6 as an average.

If the actual coefficient of friction is greater than these limits, the arch will act as if it were solid. If it is smaller some sliding will be started. However, a sliding will reduce the shear stresses considerably without changing the radial stresses very much. The sliding therefore will stop as soon as the shear stresses are reduced to equivalence with the frictional forces. In Table 2 the author has found that for this particular arch the lamination creates a 19% reduction in stresses if there is no friction. With a coefficient of friction of 0.5, used by the author, the actual stress reduction is obviously zero, or nearly so. If a coefficient of friction equal to 0.2 could be obtained, a stress reduction of about 10% could probably be depended on, since the average limit corresponding to no reduction is 0.43.

In comparing the stresses it should also be remembered that the yielding of the foundation reduces the maximum compression in the arch, and that the percentage reduction is greater for thick arches than for thinner or for laminated ones.* This correction is not included in Table 2, which, therefore, is not quite fair for the solid arch.

Asphalt can be obtained in a number of grades by admixing with oil; all these grades are liquid if warmed, stiffer if cooled, and brittle and hard if sufficiently cold. Fig. 12 shows how asphalt used as a seal in a water-stop of a dam, has been acting as a liquid. The square opening for the asphalt seal was placed too close to the face of the dam, and the pressure from the semi-liquid asphalt broke the corner between the seal and the face. The writer was informed that this happened because of the summer heat and not by reason of artificial heating. The pressure was developed too fast to be released by slow leakages. This does not mean that the same asphalt could not run through narrow openings. At another joint the writer saw asphalt which had flowed 40 ft. from the up-stream face, through a contraction joint with an 0.03-in. opening at the down-stream face. For a part of the way it may have followed the grouting pipes, but the openings to and from these pipes are generally not wider than the opening of the joint, and at the down-stream face the asphalt was coming out of the joint and not through the pipes (see Fig. 13). The same asphalt was brittle and hard in winter.

* Compare the writer's discussion on "Stresses in Thick Arches of Dams," *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 554, particularly Tables 11 and 12. Later, the basic formula is checked by the Stevenson Creek Test Dam results, see *Proceedings*, Am. Soc. C. E., Pt. 3, May, 1928, pp. 124–129.

A low coefficient of friction can be obtained by using a thick layer of semi-liquid asphalt, which would then give a pressure in the joint corresponding to the head of asphalt at all times. The unit weight of asphalt is about the same as water, and this pressure will be, even with rather slow-flowing plastic asphalt, about the same as water pressure for the same head. All open drains would then obviously have to be omitted. However, the head of liquid in the joint would have to be controlled, since otherwise the upper lamina would be exposed to an immense pressure in the up-stream direction for empty reservoir, and, if this pressure does not open up leakages so that it can be released, the upper lamina would probably be damaged. Eventually it would collapse because it can only stand small forces in the up-stream direction. Consider Fig. 12 which shows a similar bursting on a small scale; the upper lamina in the dam would be exposed to a similar pressure all over the area of the joint instead of the pressure on a 5-in. strip at the asphalt-seal in the photograph.

Furthermore, with less than complete emptying of the reservoir a semi-liquid asphalt in the joint could do harm by wedging the two laminae apart, thus giving room for seepage water to accumulate in the upper part of the joint. In case of freezing, this will keep the opening at the crest of the lower lamina permanent and will act as a permanent ice wedge also in case the water later is raised. In this way great extra forces can be introduced both because the laminae are wedged apart and, therefore, cannot divide the load in the right way, and because a freezing action will make the laminae act together as a solid arch.

Besides, in the ideal case, the use of a semi-liquid asphalt would divide the loads in a manner similar to that for the Dordogne River Dam. For the arch in Fig. 8, for instance, this would require a division of the thickness in $20.4 + 5.1 = 25.5$ ft., instead of $12.75 + 12.75 = 25.5$ ft. For that arch element this would again eliminate practically all the gain from the lamination since the thicker lamina is almost as thick as the solid arch.

The author does not seem to plan on the use of any liquid or semi-liquid asphalt, and this discussion is only inserted to show that, if one tries to obtain a low coefficient of friction in that way, it is better to adopt a system of controlled filling with water as was done with the Dordogne River Dam.

The question then is whether coefficients of friction less than 0.5, for instance, as assumed by the author, can be obtained for the asphalted joint during the winter, when a grade of asphalt is chosen which is not liquid or semi-liquid enough to produce static pressure in the summer. The writer does not believe that can be expected; the fact is that such grades of asphalt will be almost as hard as stone in the winter. With surfaces made smooth by using steel forms, coefficients of about 0.5, and perhaps as high as 0.7, are probable. The lamination will then be of little or no use for equalizing the arch stresses. With coefficients as low as 0.1 to 0.2, the lamination will be of some use even if it will not reduce the stresses as much as the author has assumed, unless the coefficient of friction is zero.

The author has computed the relative movements at the joint, using a modulus of elasticity of 3 000 000 lb. per sq. in. According to the tests made

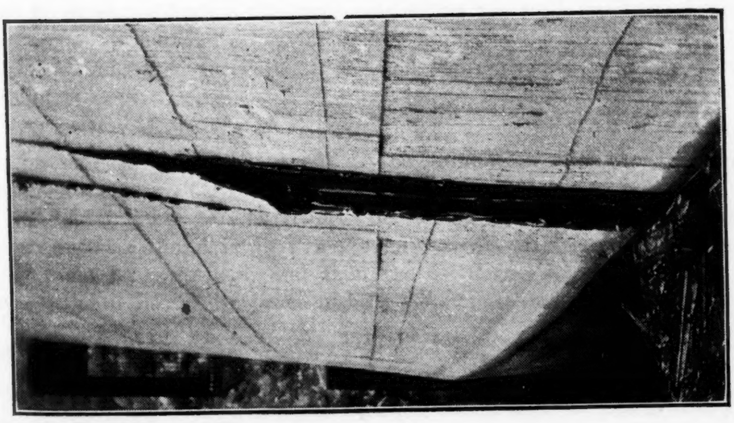

FIG. 12.—BLOWOUT DUE TO STATIC PRESSURE FROM SEMI-LIQUID ASPHALT.

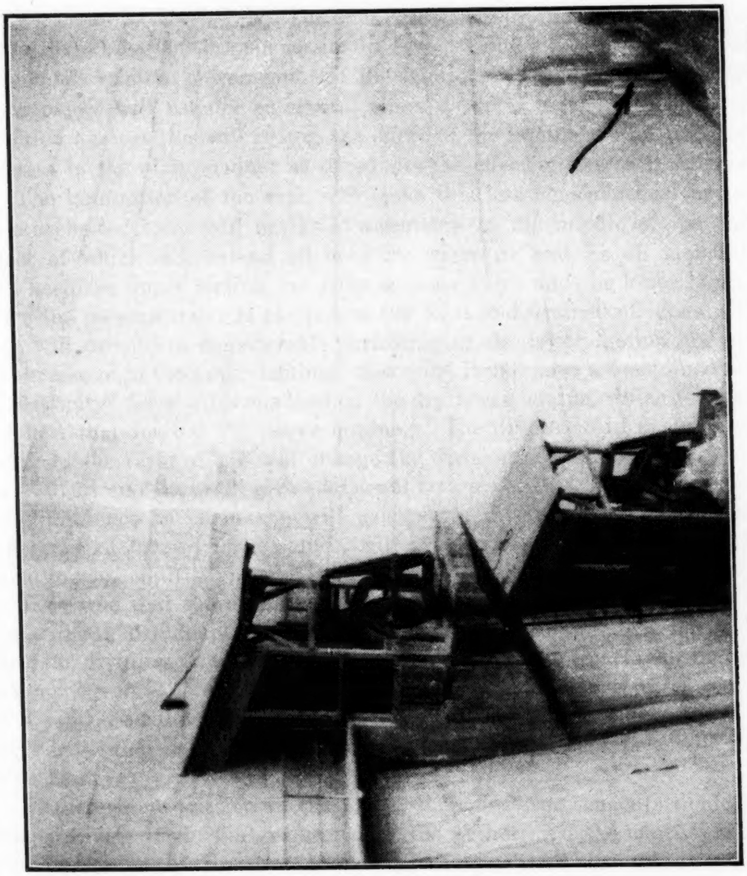

FIG. 13.—ARROW SHOWS PLACE AT WHICH ASPHALT WAS LEAKING OUT AT DOWN-STREAM FACE FROM WATER-STOP AT UP-STREAM FACE AND THROUGH NARROW CONTRACTION JOINT.

by Raymond E. Davis,* M. Am. Soc. C. E., it does not seem fair to assume more than 1 500 000 lb. per sq. in. in case of long time loadings, which will double the relative movements. If the friction is zero, the vertical sliding at some places will usually be several times as great as the horizontal sliding. Friction against this will reduce the effect of the lamination on the cantilever stresses in the same manner as it reduces the effect on the arch stresses.

The lamination of the arch will make this less homogeneous than before. The asphalted joint will act as a water-stop in the middle of the dam, the effect of which will extend all over the structure and, in all probability, it will keep the upper lamina wet from seepage water and the lower lamina dry, since the seepage water is stopped at the joint and drained off there. This at first will produce a considerable shrinkage in the lower lamina and little or no shrinkage in the upper lamina. Secondly, it will gave a "sustained modulus of elasticity" for continuous load on the dry lower lamina, which is only about one-half that for the wet upper lamina.† The difference in modulus of elasticity in the ratio of 1:2 will change the division of load from 50 + 50% to 33 + 67%, and thus will give additional stresses of 33% for the upper lamina. The difference in shrinkage will exaggerate this condition considerably; a difference of 0.02%, for instance, will introduce another 150 lb. per sq. in., equal to 25% additional stress if based on 600 lb. per sq. in.

It is true that there will be some effects of this kind also in solid arches, but without any doubt the use of a water-tight membrane in the middle of the dam, dividing this into an upper wet and a lower dry part, will increase the non-homogeneity, and thus will increase such additional stresses materially. In a solid arch the water-soaking will be rather uniformly distributed due to capillarity and water pressure; only at the down-stream face will the concrete be dry.

If there is no friction in the joints, the cantilever elements would become comparatively ineffectual, as stated by the author, and the cantilever stresses would be reduced. However, it should be remembered that this is not always desirable. For instance, for the Gibson Dam,‡ the arch stresses would no doubt be much too high if the cantilever action was made ineffectual, and this therefore would demand a heavier arch section.

Conclusion.—The foregoing discussion leads directly to five conclusions, as follows:

(1) With a coefficient of friction of 0.5, as assumed by the author, the friction will eliminate the benefit of the lamination either entirely or to a very great extent.

(2) The friction can be reduced by using liquid or semi-liquid asphalt grades, but if a liquid is used in the joints, even if it is rather viscous, the filling must be controlled, and then the Dordogne River Dam with water between the arches would prove to be a better pattern.

* *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 213; and *Transactions*, Am. Soc. of Testing Materials, 1929, Pt. II, p. 678.

† See tests by Professor Davis, *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 213; and particularly *Transactions*, Am. Soc. for Testing Materials, 1929, Pt. II, Fig. 19.

‡ Constructed by U. S. Bureau of Reclamation, Div. of Load, see *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1295.

(3) The asphalted joint will make the concrete non-homogeneous (different moisture content in the laminae), and this will introduce additional stresses which are much greater than the reduction which could be obtained by lamination in the case of no friction.

(4) The writer believes that, in most cases, it is desirable either to avoid gravity wings and buttresses or to reduce their height by using a longer span for the arch than is shown in Figs. 5 and 7. The most favorable height of gravity wings can only be found of course by comparative designs.

(5) The use of over-hang of the crown cantilever is believed to be a great advantage where the constant-angle arch principle can be better accomplished in that way.

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PAPERS AND DISCUSSIONS

BALDWIN FILTRATION PLANT, CLEVELAND, OHIO
Discussion*

By Messrs. S. M. Van Loan, and Harry N. Jenks.

S. M. Van Loan,† M. Am. Soc. C. E. (by letter).‡—This paper has been written in a most interesting manner and the description of the plant, together with the tabulated costs, is of high value to all construction engineers.

The writer has been impressed by the structural beauty of the exterior and interior of the Baldwin Filtration Plant. During a visit in 1926, his interest centered upon the influent to the coagulating basins. At the time, the hydraulic-jump mixing flume was under operation. It would be interesting to know whether this installation has given the results which were hoped for and which were expected, and if it bears out in actual operation the expectations that were fore-cast by the experimental flume.

In June, 1926, the filter beds were operating under a burden of organic growth, and numerous cyclops were noticed on the sand where water was below the surface. Has much trouble been experienced in the operation of the beds due to the organic growth in the applied water? Have taste-producing substances been injected into the lake water so as to contribute either a taste or an odor to the influent of the Filtration Plant, and, if so, what chemical applications have been made for their reduction or elimination? Information has reached the writer indirectly that experiments with ammonia have been made in Cleveland, and it would be of interest to know how far this research work has been carried and what assumptions have been noted.

A reply to these operating questions will be of great value to the filter plant operator regardless of the type of plant on which he may be engaged.

Harry N. Jenks,§ M. Am. Soc. C. E. (by letter).‖—The authors have placed the Sanitary Engineering Profession greatly in their debt for the

* This discussion (of the paper by J. W. Ellms, G. W. Hamlin, A. G. Levy, and J. E. A. Linders, Members, Am. Soc. C. E., published in February, 1930, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Deputy Chf., Bureau of Water, Philadelphia, Pa.

‡ Received by the Secretary, February 28, 1930.

§ Cons. San. Engr., Eng. Office of Clyde C. Kennedy, San Francisco, Calif.

‖ Received by the Secretary, May 16, 1930.

wealth of technical data and interpretative comment which they have furnished. In discussing a paper of such scope, one is naturally inclined to limit one's self to a consideration of certain specific topics, because only those intimately acquainted with the design conditions and requirements are in a position adequately to evaluate the merits of the conception of the filtration project as a whole.

Following the order of presentation found in the paper, it is significant to note the prominence given the hydraulic data and the influence that hydraulic values have on the the satisfactory operation of the plant itself. In particular, the proper control of velocities of flow throughout the plant is considered of great importance to the success of each stage of the process.

One of the most important elements in successful coagulation is the provision of adequate mixing velocities, and the subsequent preservation of the floc, so essential to satisfactory sedimentation.* A distinguishing feature of the mixing stage in the Baldwin Filtration Plant is the hydraulic-jump flume installation. The value of this initial rapid mixing is rightly becoming more generally recognized. The writer has obtained excellent results in coagulation by passing the water through an aeration system with a small dose of alum to produce a pin-point floc. Thorough coagulation ensued rapidly thereafter when the remainder of the total dose of alum was added. Under these circumstances, also, the aerators served the useful purpose of dissipating the quota of carbon dioxide resulting from the application of the initial dose of coagulant.

From Table 1† it is noted that the observed loss of head through the flume is 2.40 ft. This is in favorable comparison with that encountered in baffled basins or channels for an equal degree of mixing. From the basic data submitted, it is seen that the velocity of the water in the flume is 12.5 ft. per sec. The initial mixing period is, therefore, at most only a matter of a few seconds. The question one might raise is whether full returns for the loss of head of 2.40 ft. are obtained under this time limitation. It seems that a more commensurate initial jarring of the water might be obtained through mechanical mixing. The flume, however, is a very compact device and one that requires no operating attention. Would it not also be possible to construct it so as to make it a means of measuring the flow, in the manner of the Parshall flume?

The use of mechanical mixing devices is deservedly gaining favor among designing engineers and operators of water treatment plants. It is generally desirable to provide for decreasing mixing velocities, which may be obtained through the use of stirring mechanisms operated at corresponding variable speeds. In one of his installations the writer introduced an original design which he has termed "spirovortex" mixing, wherein a controllable mixing velocity is maintained in the coagulation tanks by means of the recirculation of a part of the flow by pumping. Results of experiments indicate the desirability in some cases of providing for chemical agitation as distinguished from

* "Elements of Successful Coagulation and Filtration," *Water Works,* Vol. LXVII, No. 5 (1928).

† *Proceedings,* Am. Soc. C. E., February, 1930, Papers and Discussions, p. 206.

mixing as such, to which the "stream-flow" method of recirculation over a riffled surface is particularly well adapted. This in effect involves a series of small hydraulic jumps.

Intermediate between the well-ordered mixing of coagulation and the semi-quiescence of sedimentation, a period of very gentle stirring or streaming should be interposed to produce agglomeration of the floc. This takes place at velocities ranging, for typical waters, from 0.1 to 0.15 ft. per sec. The tank or channel capacity required for this period is more than offset by the resulting possible decrease in the size of the sedimentation basin.

It is significant to note the authors' care in providing for sufficiently low velocities in the coagulated water channels to prevent the destruction of the floc. It is true that an endeavor should be made to keep the velocities within the range of 1.5 to 2.0 ft. per sec. The writer is inclined to the belief, however, that it is not so much the specific velocity as the character of the flow itself, which is the governing factor. It seems that the floc may be unharmed even if considerably higher velocities are present, provided the flow is smooth and free from excessive eddies. Abrupt changes in direction, causing impact, represent the most unfavorable conditions. Furthermore, the entrainment of air, or the agitation of the water by air, is conducive to the destruction of the large flocs, and their re-solution into a colloidal state. In general, channels and gate openings which have good hydraulic conditions involving low hydraulic losses are favorable to the preservation of the floc, especially if the hydraulic radius of the channel is large, thus inducing a relatively small amount of "drag" on the side walls in comparison with the volume of flow.

From Table 1 one may note the close agreement between the computed and observed loss of head through the pre-treatment portion of the Baldwin Filtration Plant. Such information should be of value to designers in making an appropriate allowance for such loss. In this connection Fig. 19 will be of interest. This is a balanced curve and is drawn through the plotted points of observed losses of head. The nominal design of capacity of the plant to which this curve refers, was 32 000 000 gal. per day, a rate at which the total loss of head is seen to be 1.8 ft., increasing to 5.0 ft. at the designed maximum flow of 48 000 000 gal. per day.

During recent years it has become apparent to sewage works engineers that optimum conditions for sedimentation are more dependent on basin area than on crude detention period. Advantage is now being taken of the important influence of the area factor in the design of relatively economical, shallow basins for water filtration plants, the basins being provided with continuous sludge removal. Current investigations by the writer indicate that hydraulic means for the satisfactory removal of the sludge may be applied on a large scale.

The authors' data show that the following significant factors entered into the design of the Cleveland coagulation basins:

Total net settling area (inside of basins), in square feet..	283 200
Average depth of water, in feet........................	15
Velocity of flow, in feet per second (at 2.36 ft. per min.)..	0.04
Detention period, in hours............................	4.7

Based on a flow of 165 000 000 gal. per day, or 114 500 gal. per min., the area allowed for sedimentation equals 2.47 sq. ft. per gal. per min., which is a high enough value for this factor to account for the excellent sedimentation efficiency found at the Baldwin Plant. Since the volume of flow is 917 000 cu. ft. per hour, the conditions of operation permit an overflow rate of 3.25 ft. per hour. This appears to be quite conservative for alum floc.

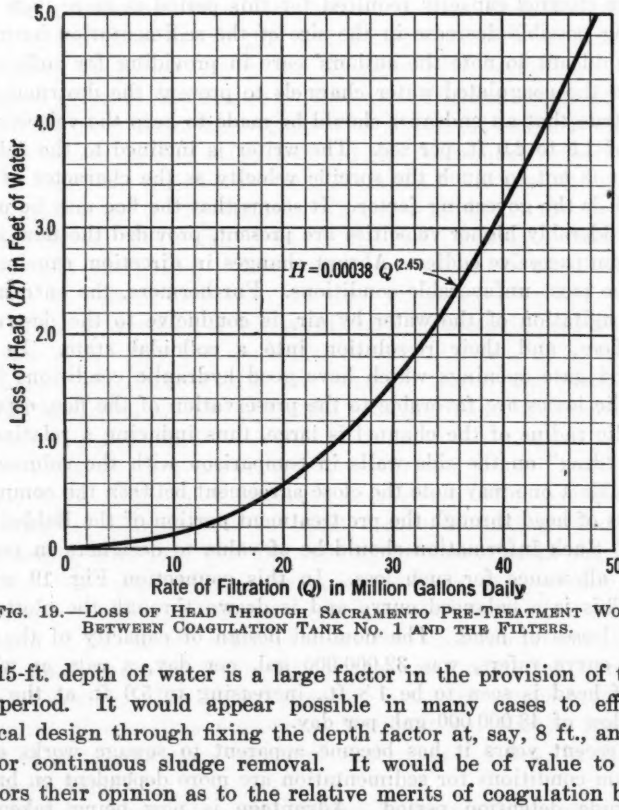

FIG. 19.—LOSS OF HEAD THROUGH SACRAMENTO PRE-TREATMENT WORKS, BETWEEN COAGULATION TANK NO. 1 AND THE FILTERS.

The 15-ft. depth of water is a large factor in the provision of the 4.7-hour storage period. It would appear possible in many cases to effect a more economical design through fixing the depth factor at, say, 8 ft., and installing means for continuous sludge removal. It would be of value to learn from the authors their opinion as to the relative merits of coagulation basin design predicated on the overflow rate theory rather than the detention period, under conditions existing at the Baldwin Filtration Plant.

It is interesting to observe that the sedimentation velocity provided by the designers of this plant is 0.04 ft. per sec. rather than a substantially lower velocity which is often found in basins constructed along less economical lines. In general, each water has a critical velocity of subsidence, below which it is unnecessary to go. A further consideration relating to velocities of sedimentation basins merits more attention, perhaps, than it has received among designers. The writer's operating experience leads him to the conclusion that, in general, preference should be given to a structural arrangement whereby gradually decreasing velocities may be secured through the sedimenta-

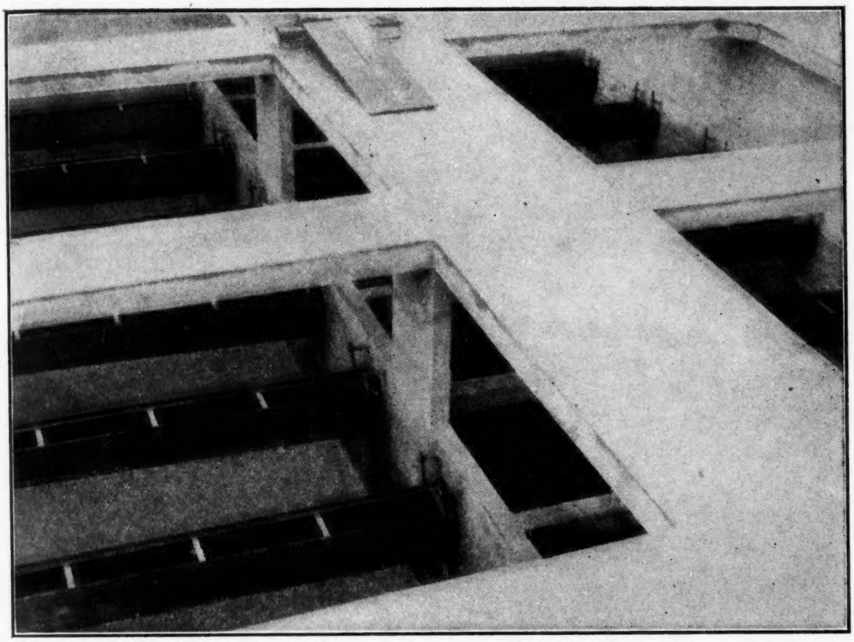

FIG. 20.—VIEW OF WASH-WATER GUTTERS IN SACRAMENTO FILTRATION PLANT, ADJUSTABLE IN ELEVATION BY MEANS OF HANGER BOLTS AT EACH END.

FIG. 21.—AERATION LATERAL AND RISER SYSTEM, DESIGNED ON BASIS OF HYDRAULIC PRINCIPLES INVOLVED IN PROPORTIONING THE ELEMENTS OF PERFORATED-PIPE FILTER UNDERDRAINS, SACRAMENTO FILTRATION PLANT.

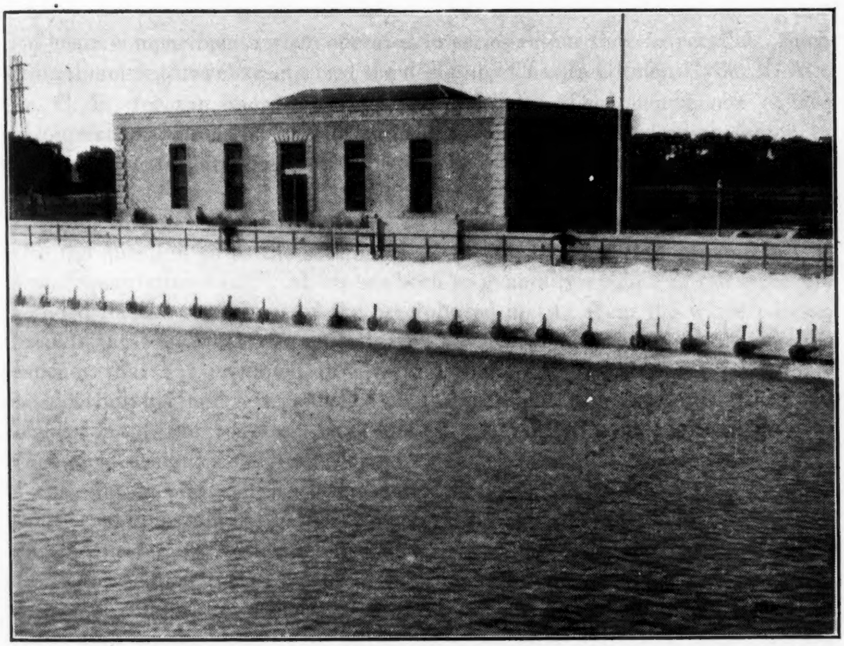

FIG. 22.—SACRAMENTO AERATION FIELD IN OPERATION, SHOWING UNIFORMITY OF DISCHARGE OVER ENTIRE AREA, THROUGH FLOATING CONE NOZZLES MOUNTED ON PIPING SYSTEM OF FIG. 21.

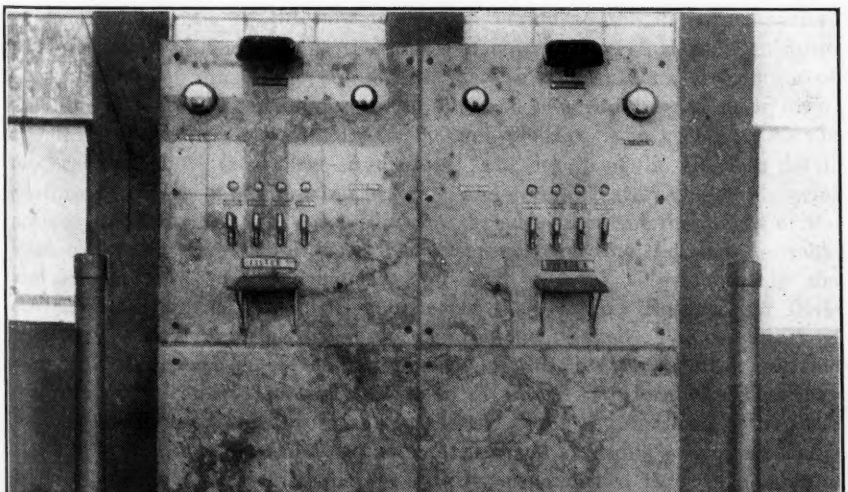

FIG. 23.—ORIGINAL ELECTRICALLY OPERATED FILTER CONTROL PANEL BOARD INSTALLATION AT SACRAMENTO, CALIF.

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tion basin compartments when operated in series rather than in parallel. Such a functional feature characterized the design by Charles Gilman Hyde, M. Am. Soc. C. E., for the Sacramento Filtration Plant. The phenomenon of floc agglomeration, which is thus brought into play, is an important factor in rapid sedimentation after the subsiding velocity for the floc is finally attained.

While discussing the subject of sedimentation, it seems appropriate to raise the question as to the soundness of the theory of design implied by the term, "coagulation basin", which has been so generally applied to the structure intended for the settlement of the precipitated matter from the water passing through the basins. It is a matter of considerable surprise to observe the tendency that has prevailed in filtration plant design, involving inadequate chemical mixing and a dependence on the sedimentation basin for the actual completion of the required reactions of coagulation. In the interests of economical design, as well as economy and flexibility of operation, the stages of coagulation and sedimentation should be complete in themselves, and distinctly separate from each other. When means are provided for thorough coagulation (preferably with agglomeration), it will generally be found that the size of the sedimentation basins as such may be reduced to a degree that will more than compensate the cost of prolongation of the chemical mixing period. Then with proper inlet and outlet arrangements and with long, narrow basins, similar to those at the Baldwin Plant, it is possible to obtain exceptionally high sedimentation efficiency, as measured by the removal of suspended solids and the ratio of the flowing-through period to the detention period. The deposition of sludge along the course of flow is also remarkably uniform.

A number of interesting points are raised in the author's description of the design features pertaining to the filters. The capacity of each filter, amounting to a little more than 4 000 000 gal. per day, is in keeping with the modern tendency toward large-sized units. The design of the filters in detail conforms to the best engineering practice, and several features deserve special notice. Among these is the provision for the adjustment of the height of the wash-water gutters.* Much is yet to be learned regarding the most desirable and efficient rates of wash, and size and depth of sand, as evidenced by the current investigations of the Committee of the Sanitary Engineering Division on Filtering Materials for Water and Sewage Works.† The possibility of changing the elevation of the gutters enables the operator to meet variable operating requirements, as well as to take advantage of information that may become available in the future as regards the physical functions of the filter itself. At Sacramento, Calif., adjustment is made by bodily raising or lowering the entire gutter, which may be readily accomplished through manipulation of the hanger-bolts shown in Fig. 20. At Fort Dodge, Iowa, the writer provided similar adjustable gutters, primarily with a view to enabling appropriate modifications to be made whenever a change from deep wells to the Des Moines River as a source of supply should become necessary.

* *Proceedings*, Am. Soc. C. E., February, 1930, Papers and Discussions, p. 218.

† *Loc. cit.*, September, 1929, Papers and Discussions, p. 1799.

The authors describe a strainer system designed in accordance with results of experiments by Mr. Ellms.* There are two typical situations of this kind in a filter plant, (1) the wash-water manifold and laterals in a filter; and (2) the header and pipe system for an aeration field. The latter is illustrated in Fig. 21. One of the authors has shown† that the hydraulic theory applicable to the design of the filter underdrain system (laterals and perforations) may be used successfully in proportioning manifolds and laterals. The validity of this theory is well attested by the operation of the Sacramento aerators shown in Fig. 22. This view shows an aeration field approximately 172 by 40 ft. in plan. In this case the ratio of the combined areas (a) of pipes to the area of the main header (A) is, $\frac{a}{A} = 0.41$. The pipe laterals are 10 in. in diameter and 40 ft. long while the header channel is 12.5 by 4.5 ft. The aggregate areas of the nozzle risers in each lateral are equal to four-tenths the area of the lateral. There are 420 sprays spaced 4 ft. on centers, the discharge from which is uniform at all rates (see Fig. 22).

It is significant of the tendency toward the use of deeper gravel beds that the authors specified 22 in. of graded gravel in the Baldwin Filters.‡ The writer is also in agreement with the use of smaller sizes of rock than has often been furnished. His own experience confirms the wisdom of having a relatively large portion of the total depth comprise rock ranging in size from $\frac{3}{4}$ to $\frac{1}{16}$ in. This adds materially to the stability of the upper gravel layers and is a good safeguard against the intermingling of the overlying sand with the gravel beneath.

In connection with the filter equipment installed at the Baldwin Plant, the provision of only two gauges for each filter, namely, the wash-water and loss-of-head gauges, is in reality all that is required. From the standpoint of economy durability, and attractiveness, one may regard with favor the development of filter operating tables in the form of electrically-operated panel boards. The first installation of this type was made under the writer's direction at Sacramento,§ and more recently it was introduced in the water-works design at Fort Dodge, Iowa. Fig. 23 shows the original Sacramento panel board. The provision of electrical indicating instruments obviates the requirement of a specific position of the control panel to fit the location of float tubes and accessories; also, all actuating strings and rods can be eliminated, particularly on the valve-opening indicators, which are readily actuated by reactance coils responding to the movement of the valve stem followers.

* *Proceedings*, Am. Soc. C. E., February, 1930, Papers and Discussions, p. 220.

† *Journal*, Am. Water Works Assoc., Vol. XVIII, December, 1927, p. 664.

‡ *Proceedings*, Am. Soc. C. E., February, 1930, Papers and Discussions, p. 219.

§ "Electrically-Operated Gages Devised for Filters", *Engineering News-Record*, Vol. 95, No. 9 (1925).

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

RELATION BETWEEN RAIL AND WATERWAY TRANSPORTATION: A SYMPOSIUM
Discussion*

By Messrs. Baxter L. Brown and Theodore Brent.

Baxter L. Brown,† M. Am. Soc. C. E. (by letter).‡—In accepting the Hoover Medal on April 8, 1930, President Hoover stated:§

"With the development of our great national tools, our engines, our railways, our automobiles, our steamships, our electric power and a score of other great implements, together with the supplies of material upon which they depend, the engineer has added vastly to the problem of government, for government must see that the control of these tools and these materials are not misused to limit liberty and freedom, that they advance and do not retard equality of opportunity amongst all our citizens. * * * But when the problems which these great tools create come to the door of government, they are at once emotional problems, * * *. Our greatest difficulty in dealing with these problems of government is when the emotion comes first. Facts and the technical knowledge come but slowly or are often lost in a sea of embittered controversy."

Transportation has been one of the greatest problems of mankind since the earliest days and will so continue until the end of time. Competition of the various methods of transportation is the best incentive for its constant improvement, but such competition must be along fair and just lines, or it will be "misused to limit liberty and freedom" and "retard equality of opportunity amongst all citizens." It was this thought that caused the passage of the Sherman Act.

Owners of the best and shortest routes, where the cost of transportation is naturally less than by any other route, should be protected in their advantage

* Discussion on the Symposium on Relation Between Rail and Waterway Transportation, continued from May, 1930, *Proceedings*.

† Cons. Engr., St. Louis, Mo.

‡ Received by the Secretary, May 1, 1930.

§ *Mechanical Engineering*, May, 1930, p. 528.

and allowed to profit by their foresight, in being the first in the field, or, by their sagacity, in securing the best available facilities. Obviously, they will get the "cream" of the business, and competing lines must be satisfied with the less desirable class of business, and they must handle it for a smaller margin of profit. It would be palpably unjust for the Government to interfere and endeavor to reduce or remove the natural advantage enjoyed by the owner of the best system, so that others could continue in operation.

"Equality of opportunity" does not mean that any individual should refrain from exercising his talents for his own benefit, in order that another person less fortunate in ability should have a better chance to acquire the desirable things in life; nor, having possession of these better things, whether acquired through his own exertion or by inheritance, is it necessary for an individual to relinquish these advantages, for the benefit of others less energetic or less far-sighted than himself or than those that have gone before him.

The daily newspapers of April 26, 1930, reported that the House of Representatives had passed the River and Harbor Bill, carrying $15 000 000 for improvement of the Missouri River between Kansas City, Mo., and Sioux City, Iowa. It is questionable whether all the freight carried on the river between these two points during the period, 1930 to 1950, will be worth that much. Does any one believe that a proposition to spend that amount of money on that stretch of river for the purpose of putting it into condition to carry freight in competition with the railroads could be put before any group of financiers or bankers with the slightest hope of getting any favorable consideration?

There seems to be a disposition to bolster up unprofitable enterprises by subsidizing them by the Government, State, or city. From the standpoint of economical and efficient management, there is no reason for the existence of any institution, outside those classed as charitable, which is not self-sustaining.

Upon whether one views the relation between rail and Mississippi River transportation from the emotional or from the clearly commercial standpoint, depends the answer as to whether river transportation, as exemplified by the Government's experiment under the name of the Inland Waterways Corporation,* is a success or a failure.

This governmental agency is engaging in unfair competition in the attempt to demonstrate that it is a commercially profitable enterprise. In the preparation of financial statements to be used in support of this argument, no consideration is given to certain expenditures which would necessarily be included by a privately owned enterprise, such as interest on investment, and taxes. The enterprise is profitable only to those who are so fortunately situated as to be able to take advantage of its arbitrary rate of 80% of the rail rate.

An examination of the published records of the Corporation shows that 1928 was apparently the best year, when a net income of $327 712 is indicated. If, in this year, the Corporation had been required to pay taxes in the same ratio as the railroads (6.37% of their gross revenues) its expenses would have

* *Proceedings*, Am. Soc. C. E., March, 1930, Papers and Discussions, p. 546.

been increased $427 272. If interest at 5% had been charged on the depreciated value of its equipment, there would have been an additional charge, to expenses, of $785 647, making a total of $1 212 919. This indicates an actual loss of nearly $900 000.

The Corporation's annual report for 1923 states that Congress had appropriated, up to 1923, nearly $750 000 000 to create navigable streams and canals. It is not stated what portion of this could properly be allocated to the channels used by the Corporation. Interest on this sum at 5% is $37 500 000, a substantial part of which is properly chargeable to the Corporation's operation and enters into the cost of transportation furnished. It is paid by the taxpayers as a whole and not by the shippers using the service.

No part of the maintenance of the channels used is charged against the Corporation. This expense is unknown, but if it is assumed that a proper charge for this feature would be in the same ratio as the Maintenance of Way and Structures expenditures for Class I railroads for 1928 (13.71% of gross revenues), the Corporation's expense would be increased $919 608.

Attention is called to the loan of $880 000 to municipalities and the fact that the terminals of the Corporation have been in large part furnished by the municipalities served. The Corporation's Chairman, before the Appropriations Committee, mentions fourteen cities that have expended a total of $24 000 000 in providing terminals.

It is evident, therefore, that the actual costs of operating the Corporation for 1928 are under-stated to the extent of at least $1 800 000. If the traffic had been handled by rail at charges 25% in excess of those of the Corporation, the increase in cost would have been $1 677 000, or a net saving to the public of $123 000, exclusive of interest charges on funds used to provide channels.

The public, not the shipper, assumes these omitted costs, and, on a commercial basis, they must be considered in determining the cost of transportation furnished. As nothing is included for profit in this comparison, the prospect of persuading a private corporation to undertake the operation is not encouraging.

There has been a growing tendency to put the Government into business, from the operation of railroads to the introduction, in the present Congress, of an Act to permit engineers of the Bureau of Public Roads to serve Latin-American Governments while remaining under full pay by the United States Government. This is a case of proposed subsidizing of engineers in direct competition with legitimate business and is parallel with the Government's subsidizing of waterway transportation.

The Government's experiment in river transportation has reached a point that changes the problem from one of transportation to the larger and broader one of the Government's engaging in business in direct competition with private enterprise. Is the foundation being laid for the Nation to become another Russia, with all that the name implies, or is the United States Government to continue dedicated to the protection of "life, liberty, and the pursuit of happiness?"

THEODORE BRENT,* Esq. (by letter).†—The general impression gained from the paper by Mr. Cornish is that the entire subject has been viewed by him with conservatism.

Concerning the demands of the country for transportation, the general rule of calculation has been that traffic demands double every twenty years, on the basis of the present growth of the United States. This only calls for a sustained increase of 3.5% annually. Despite the dislocation and inequality of the World War period, this rule has been quite well justified in the elapsed period of the decades, 1910 to 1930, and there are no present signs of a cessation of normal National growth.

The calculations of a public requirement of 870 000 000 000 ton-miles of transportation by 1950 will probably be exceeded if the facilities are provided in time to permit traffic to expand normally. If the demand is not promptly met, then business must inevitably turn in upon itself and by a marked redistribution of industry and manufacture permit of such growth by a lessening of the volume of transportation between producer and consumer. The dislocation and capital expenditure involved in such redistribution must inevitably be greater than that necessary to enable transportation to keep pace.

The estimated cost of new railroad facilities to meet these requirements seems ultra-conservative. In its final form the paper calls for the production of about 50 000 additional miles of main track with yards, sidings, equipment, power, and permanent structures necessary to enable this and the existing railroad plant to carry the increased load. The estimated capital cost is in excess of $11 000 000 000, or slightly in excess of $220 000 per mile of new railroad, equipped. The average investment in the 240 000 miles of main-line railroad in use to-day (1930) is $125 000 per mile. The present average traffic density is 1 800 000 ton-miles per mile of road. With the addition of the 50 000 miles of new main track, the then existing plant of 290 000 miles of main line, equipped, has to support a traffic density of 3 000 000 ton-miles per mile. Obviously, this new construction will not be an average railroad. A cost of $300 000 per mile, or a total expenditure, equipped, of $15 000 000 000 will be more in keeping. The Pittsburgh and Lake Erie Railroad, which is the road of greatest density per mile in the country (10 700 000 ton-miles per mile), has cost $586 000 per mile. Making allowance for surplus equipment, the actual plant producing these results has cost about $400 000 per mile.

The new construction will not be where construction is cheap, but where it is most expensive, and freight traffic will have to bear the brunt of it. Other forms of transportation will increasingly drain away the passenger traffic and that which remains must become more and more expensive, without creating the ability to increase fares or service charges materially. An estimate of $15 000 000 000 is nearer the mark than $11 000 000 000 in the writer's survey of the situation.

The increase of 10% annually for river traffic for the twenty-five years following 1930 is conservative. The relatively small growth of traffic on the

* Pres., Redwood Line, Inc., New Orleans, La.

† Received by the Secretary, May 1, 1930.

Lower Mississippi River during the operation of the Federal Barge Line in the three years, 1926 to 1929, is due solely to the delay of the Government in providing needed facilities and the relatively meager power of its incomplete joint relations to attract traffic. Both these handicaps have recently been remedied and the country is due to see a marked upward trend of ton-miles at least for the 10-year period (1930 to 1940).

It is possible to measure the maximum capacity of a waterway which has limiting physical features, such as locks; but who can measure the capacity of a well-regulated river, such as the Lower Mississippi? Three tows per week in each direction between New Orleans, La., and St. Louis, Mo., produced about 1 338 000 000 ton-miles of transportation in 1928. Loaded to capacity both ways, these tows could as readily have produced 3 000 000 000 ton-miles. Tows of this size could be dispatched at half-hour intervals and be less noted than the daily parade of lake freighters in front of Detroit. Such service would produce 300 000 000 000 ton-miles of transportation twice the estimate given by Mr. Cornish and still the capacity of the river would in no manner be taxed. The estimate of the carrying capacity of this river, at least, is conservative.

The cost figures on most inland waterways are still experimental. Projecting present known costs on the Lower Mississippi to a basis of 2 500 000 000 ton-miles, the cost of operation, including depreciation of plant and terminals, is 3.27 mills per ton-mile. Profitable operation in the carriage of bulk commodities at 1 mill per ton-mile on the Lower Mississippi River will be reached some years before the inland waterways are called upon to carry burdens comparable with the 60 000 000 000 which this study forecasts.

The day of cheap railroad transportation is past. Sweeping increases in rates are being made in all important territories to bring railroad revenues to the mark fixed upon as necessary to support and expand the plant.

Shippers are only beginning to realize at what low costs good transportation by water can be performed at a profit. The cleavage between the relative cost and the value of the service increases year by year. The very force of necessity is bringing the entire rail-water transportation problem rapidly from the academic to the experimental stage. There will always be a field for controversy, but with growing appreciation of an informed public the assertions which to-day require "damnable iteration" will take the form of truisms. For the time being, however, it is necessary to go on proving, over and over again, that water transportation is cheap and rail transportation is very expensive. Mr. Cornish does this in a masterly way. This research proves the old truth in a new way and is a valuable reference work.

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PAPERS AND DISCUSSIONS

CITY PLANNING AS RELATED TO THE SMALLER CITIES

Discussion*

By Messrs. John L. Starkie and O. H. Koch.

John L. Starkie,† M. Am. Soc. C. E.—It seems to be commonly accepted that every city, irrespective of size, is subject to the same general principles in the application of a modern, comprehensive city plan. There are few, if any, problems present in the larger cities that are not also encountered—although most always in lesser degree—in the smaller communities. An economic advantage, however, rests distinctly with the smaller city, in that, the earlier in the state of development of a city the plan becomes operative, the greater will be the saving in cost and the simpler the task of the city planner. In meeting an incurable obstacle in the preparation of a city plan or one surmountable only at prohibitive cost, what engineer has not dreamed of the ideal condition in which a city has waxed and expanded from the date of its inception in accordance with a well prepared and well directed city plan? However, such attempts as have been made in that direction may well be termed "noble experiments".

It is fortunate, perhaps, for the progress of the science of city planning in America that the advancement of a city under planless processes must usually reach the point where the resulting chaotic and wasteful conditions awake public consciousness to a realization of the need and economy of an ordered and directed procedure in conformity with a carefully thought out and scientifically prepared plan. This is good because it presents a variety of difficulties to overcome which make for progress in the perfection of the science, and it enlists initially the much-to-be-desired interest of the citizen.

The problem of securing and holding the interest of a larger percentage of the citizens is usually less of a task in smaller cities. However, there is

* This discussion (of the paper by E. A. Wood, M. Am. Soc. C. E., presented at the meeting of the City Planning Division at Dallas, Tex., on April 25, 1929, and published in May, 1930, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Brownwood, Texas.

more of a tendency to urge the engineer to incorporate in the plan ideas that are supposedly advanced for the alleviation of existing faults, but with no outlook on the future.

Many merchants in the smaller cities regard a traffic congestion on the streets in the vicinity of their places of business as a sure sign of progress and a condition to be preferred. Fortunately, however, highway engineers in Texas are insisting on locations from a broader viewpoint, so as to by-pass through traffic around population centers and, at the same time, to avoid railroad grade crossings wherever possible.

The traffic problem is present everywhere, in the small as well as in the large city, and the elimination of railroad interference with thoroughfares occupies a very important part in considering that question. From a long experience in railway service the speaker has no hesitancy in saying that practically every reasonable request, equitably made, to share in the expense of grade-crossing separations on existing locations of railways will meet with ready and co-operative response on the part of the railroad so far as financial abilities will permit. There is an alarming increase in the number of new grade crossings being opened, as compared with the number that are being eliminated. The city planning engineer who specifies, with every railroad grade-crossing separation which he proposes, the closing of one or more existing grade crossings, serves humanity in general as well as the city which employs him.

A statement by the Master Plan Engineer of the City of Dallas that "planning should be done along regulatory lines rather than the correction of existing errors", indicates an active interest in the whole success of city planning. The 1927 Enabling Act of Texas also takes cognizance of this viewpoint. Any city plan which proposes in whole or in part for immediate consummation, changes, alterations, or relocations, at a wholesale rate, is untimely. A city plan once adopted should remain in operation perpetually (subject to such modifications as changing conditions demand), and the accomplishment of its objectives should be made a gradual process with its complete realization possibly never fully attained. Thrifty railroad companies are continually changing their standards of various physical properties, but, except in the rarest of instances, the new facilities are not installed until the old ones have completed their useful service life, or until unusual economic conditions warrant a change. Adaptation of this thought to city planning is illustrated in the "set-back building line". The process may be slower, but the transition to the accomplishment of the plan is performed in an orderly manner; it interferes least with the even tenor of the industrial, commercial, social, and moral life of the community, and, in the end, it provides a more enduring continuous and useful service to the greatest number of citizens.

O. H. Koch,* M. Am. Soc. C. E.—That city planning is just as vital and important to the smaller cities as to the larger ones has been emphasized by the author. He has outlined a number of principles and recommendations as regards city planning in general. The speaker would like to call attention

* Dallas, Tex.

to a few points wherein planning in the smaller cities differs from that in the larger cities.

A population of less than 50 000 may be considered to define a small city. This is the type that predominates in Texas as well as in other States. Texas has many cities of less than 50 000 inhabitants, but only a few that are larger. The same type of discussion could be very readily carried on as to the difference in planning between cities of 100 000, or 200 000 inhabitants and the great metropolitan centers. In each of these various classes there are many local and individual problems which are peculiar to each classification, and which need to be solved in a different manner. Many people have the impression that city planning, in a town of 25 000 to 50 000 inhabitants, is a comparatively simple matter, but the speaker maintains that a good, sound, sensible, and practical city plan for a town of this size requires a great deal of study and careful consideration.

The first step in preparing a city plan is to take an inventory of the existing conditions. This step in a small city is, of course, much easier and more simple than in the larger cities. The next step is to analyze the projected needs of the community by studying its past record of growth and to attempt to predict the future growth and needs. In this prediction it will be necessary to determine the probable type and character of the future of the community. This problem of trying conscientiously to predict the future trend of growth has caused the speaker more concern in plans for the small city than it has for the large one, and that particular decision is one of the most important essentials in preparing any city plan. City planning should be at least 90% preventive and 10% corrective. The accuracy with which the prediction of future growth is made will determine, to a great extent, the ultimate value of that city plan.

It is relatively easier to make such predictions in the large cities because of the availability of a more definite record and tendencies of growth upon which to base predictions. In the small cities this particular information is somewhat limited, and the planner has, as it were, a shorter base line from which to project and predict future tendencies. In the large, more established cities, the trend of growth and type of development is more firmly established and its general character is more permanently fixed. Any unusual development would not change its established characteristics to any great extent compared with the material effect that the same unusual development might have upon the smaller city where the proportion of importance would be much greater. In this type of city it is possible, by the introduction of a large project, to change the entire character of its development.

For example, in a small city, supported primarily by educational institutions, and which has been residential in nature for many years, a satisfactory city plan could be developed based upon such conditions. Suppose that a large manufacturing company should decide to build its factory in this city and to import a great number of factory employees. This unusual development in such small city would tend to unbalance the entire situation; it

would change its character and would require a complete change of the plan. In the larger city the introduction of a fair-sized manufacturing plant would not make any appreciable change, since it would probably not be the first manufacturing plant in that city. That phase would have been anticipated to a certain extent, and the city plan could be adapted to meet the situation without materially affecting its character.

Thus, in a smaller city, it is necessary to decide whether or not it would be advisable to prepare a plan providing for such possible industrial developments, thereby furnishing incentives and encouragement for securing additional activities for the city, or whether it would be better to ignore such possibilities and prepare a plan which is limited to developments of its present type, thereby discouraging the possibility of securing such industrial activities.

Finance is always a subject of paramount importance to be considered in the city plan. While the first cost of engineering studies for a plan in the small city is not as great as in a larger city, the per capita cost would probably be more. This expense, however, is not a minor item considering the usual restricted budget of the small city. Even if the per capita cost were the same it would be much more of a burden in the smaller city. The same thing is true in the matter of corrective city planning. While the property values are not as high and the corrective measures would not total the large amounts which are reached in the greater cities, it is also true that smaller cities have less ability to pay even a smaller proportional cost. This condition is a great stumbling block and a real condition which must be faced. It requires considerable judgment in order to inaugurate and execute what would be thought to be a practical and economical program, both from the standpoint of first cost as well as that of executive cost.

The chief value of city planning lies in anticipating future needs, which is a matter of budgeting the city's facilities, much in the same manner as its financial program is budgeted. If the planner is skeptical in regard to the city's growth and tries to be too conservative in planning for the immediate future only, in order to hold down cost, he will not be securing the full measure of benefit to be derived from city planning. On the other hand if he is too optimistic and plans for too many years in advance he is apt to cause exceedingly large expenditures for immediate use and to have excess of expense and unused facilities. The excess cost of the superfluous facilities, the overhead carrying charges, and the maintenance of these items, capitalized over a number of years, could very readily exceed what the cost would be with a more conservative plan, even if the additional corrective measures had to be taken after a period of years. It is this choosing of the happy combinations that will determine the comparative value which any city plan may develop.

In the larger city the cost of future corrections, after a long period of years, would be considerably more in proportion than the probable cost of such additional widening would be in a small city. This fact, together with

the greater valuations and borrowing power on the part of a larger city, would justify more liberal plans and ambitious programs than it would in the small city. For example, the cost of excessive widths of the streets in a small city might be a burden, even if such cost were limited to present expenditures for excess right-of-way widening only. Even though the paving on such streets was not unusually wide for the present, the cost of paving intersections, extra length of driveway turn-outs and cross-walks for each residence, as well as the additional maintenance for such extra paving and walks, would have to be considered. While these items may seem comparatively insignificant to planners in the larger metropolitan areas, they are a much greater proportion of the original project than in the larger city and this excess cost capitalized and put on compound interest over a period of 25 or 30 years, would pay for a great deal of corrective work at that time.

In the larger cities playgrounds, parks, etc., can be provided with much more flexibility because of the greater number to be supplied; whereas in the small cities the matter of first cost per capita again enters into the economic amount which can be spent for such facilities. In the small city the density of population per acre is considerably less than in the larger city. The facilities required for playgrounds and parks would be less and if sufficient playground space were provided, based upon a liberal estimate of the density of population, several years in the future, it would add an unnecessary immediate burden upon the city in first cost as well as in maintenance charges.

In the matter of zoning, the areas set aside for business, industrial, and residential purposes would vary considerably in their proportion to one another depending upon whether the problem related to a small or to a large city. In the small one there is not so much possibility of providing transitional property that could be readily changed to permit of the expansion of the business and industrial sections.

In the small city the matter of railroad grade elimination must be handled in an entirely different manner. The small communities are not justified in spending money to eliminate the objectional grade crossings. This problem, which is often planned in the larger cities as a major operation, must be reckoned with in an entirely different manner in the small city. These same principles are typical of the other phases of city planning.

While city planning is of great value to the smaller cities, the real value will only be obtained by a practical and common sense application of its principles heretofore developed, scaled down to the proper proportions to suit the idiosyncrasies which are peculiar to the small city. City planning in the beginning was born of necessity due to congestion and intolerable conditions in the larger cities. Most of the pioneer work in this field was developed in order to relieve such conditions. These obvious advantages and the favorable results have been made possible through city planning; and these acute and intolerable conditions first gave city planning its importance. The relief gained has encouraged the city planning principles to be applied to the

preventive phase, and has educated the public to the realization of the great advantage of planning in advance. This work has been most intensive in the larger cities and the methods and principles adopted were designed to suit conditions there. With the advantages thus favorably established the move naturally is spreading into other cities as well. The success and degree of benefit which the smaller cities will receive will depend entirely upon the city planner's ability to recognize the slightly different conditions; they will depend on his sense of proportion in adapting the principles, in a practical manner, to suit the differing conditions.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

FRANK MILLIGAN ASHMEAD, M. Am. Soc. C. E.*

Died December 7, 1925.

Frank Milligan Ashmead was born at Germantown, Pa., on February 23, 1853, the son of Samuel B. and Mary (Milligan) Ashmead. His father, Samuel B. Ashmead, a leather merchant in the City of Philadelphia, Pa., traced his American ancestry back to John Ashmead who came to Germantown, from Cheltenham, England, in 1682.

Mr. Ashmead received his early education in the public schools of Germantown and Philadelphia, afterward attending the Philadelphia Polytechnic Institute from which he was graduated in 1875.

His first professional engagements were in construction work on the Dutchess and Columbia Railroad, in New York State, and the Columbia and Port Deposit Railroad in Pennsylvania. He afterward went to the Allegheny Valley Railroad Company with which road, its branches, and successors, he remained during his future professional life. He first served as Assistant Engineer on the low-grade division, after which he was, successively, Engineer, Department of Bridges, Resident Engineer, and Chief Engineer, with headquarters in Pittsburgh, Pa.

In 1901, when the Pennsylvania Railroad Company took over the Allegheny Valley Railroad and merged it with the Western New York and Pennsylvania Railroad, designating them the Buffalo and Allegheny Valley Division, Pennsylvania Railroad, Mr. Ashmead became Engineer, Right of Way, and Assistant Engineer of the Division. He remained in Pittsburgh until 1903, when he was transferred to Buffalo, N. Y., as Assistant to the Principal Assistant Engineer.

In 1918, Mr. Ashmead's health began to fail, and by request he was transferred to Pittsburgh (to be near his children), where he continued to serve as Assistant to the Principal Assistant Engineer intermittently on problems of right of way, etc., until he retired in 1921. After this time he lived, successively, in Atlantic City, N. J., and Cleveland, Ohio, in which latter city he died on December 7, 1925.

Frank Ashmead was a man of sterling integrity, retiring in disposition and much respected and beloved by his subordinates and associates. He was well informed, not only in regard to his profession, but upon many other subjects of a philosophical and economic character. While he spent practically all his professional life with one organization, he was well and favorably known on many of the Eastern railroads.

* Memoir prepared by James H. Herron, M. Am. Soc. C. E.

He was married on December 22, 1881, to Mary Lillian Herron, of Oil City, Pa., who with a son, Charles B. Ashmead, and a daughter, Edith (Ashmead) Hutchinson, survives him.

Mr. Ashmead was elected a Member of the American Society of Civil Engineers on July 6, 1887.

GEORGE BOWERS, M. Am. Soc. C. E.*

Died February 28, 1930.

George Bowers was born on October 30, 1848, in Middlesex Village, then a part of Chelmsford, Mass., the son of Sewall and Sylvia (Fisher) Bowers. He was of English ancestry; his earliest known paternal ancestor was Michael de Bures who was seated in Dorset, England, in 1075. His earliest paternal ancestor in America was George Bowers, whose son, Jerathmell Bowers, built the Bowers homestead in 1695 on land granted from the King of England. Mr. Bowers lived in this same house many years, the home still remaining in the Bowers family, having been passed down from generation to generation.

George Bowers spent his early life in Middlesex Village and Lowell, Mass., where he attended the local schools. He started his engineering work in the City Engineer's Office in Lowell, remaining for two years before entering the Massachusetts Institute of Technology with the Class of 1875. After leaving the Institute he was employed as First Assistant Engineer on the construction of the Waltham, Mass., Water-Works. Returning to Lowell, he became First Assistant Engineer and, later, City Engineer.

In addition to his regular work as City Engineer, Mr. Bowers carried on an extensive series of experiments with driven wells to determine whether it would be possible to supply Lowell with driven-well water. He was convinced that this could be done, and his judgment proved to be correct, for since the wells were put into operation, Lowell has used no other water supply. He was the author of a number of papers on his driven-well experiments.

After leaving the City Engineer's Office, Mr. Bowers took up consulting work, which he followed until his last illness. He planned, and was Engineer during the construction of, a new driven-well water supply for Chelmsford, Mass., and acted as Consulting Engineer for The Nashua Manufacturing Company, and The American Hide and Leather Company in connection with water supply problems and the disposal of wastes. During the World War, Mr. Bowers was Engineer for a development for the United States Housing Board.

At a special meeting of the Board of Trustees of the Lowell Institution for Savings, of which Mr. Bowers was First Vice-President, the following memorial was placed on the records:

"George Bowers, our First Vice-President and Chairman of our Board of Investment, died Friday morning, February 28, 1930, at a few minutes after midnight. He was born October 30, 1848, in Middlesex Village, and afterwards lived in the old Bowers Homestead on Wood Street, built in 1695. This is supposed to be the oldest house now in Lowell.

* Memoir prepared by E. B. Carney, Treas., Lowell Inst. for Savings, Lowell, Mass.

"He was elected Corporator, Trustee, Vice-President, and member of the Board of Investment in 1904. Mr. Bowers had been City Engineer for over twenty years, but in 1911 was defeated for re-election.

"No ruler ever had his subjects more at heart, nor has a great general looked after his soldiers one bit more than Mr. Bowers did when he fought almost single-handed to have the polluted water system of Lowell changed to one of purity, and thereby saving many lives. This was his Victory of Peace.

"He was faithful to all the duties imposed upon him of whatever nature, and this bank will turn back its pages to his record and say with pride, 'Well done Thou Good and Faithful Servant'."

A life-long Unitarian, George Bowers devoted time and money to help his church. He was a member of the Boston Society of Civil Engineers, the New England Water Works Association, the Engineers Club of Boston, the Vesper Country Club, and a number of historical and philanthropical societies.

On October 24, 1878, Mr. Bowers was married to Estelle L. Wilkins, who survives him. He also leaves a daughter, Helen E. Bowers, and two sons, George W. Bowers, Assoc. M. Am. Soc. C. E., and Alton R. Bowers.

Mr. Bowers was elected a Member of the American Society of Civil Engineers on October 1, 1902.

PETER FRANKLIN BRENDLINGER, M. Am. Soc. C. E.*

Died December 16, 1929.

Peter Franklin Brendlinger was born on January 18, 1850, in New Hanover, Pa. He was the son of Frederick Brendlinger, a land owner and farmer, storekeeper, and Postmaster for many years, and, at one time, County Treasurer. His mother was Mary (Hill) Brendlinger who, like her husband, was of sturdy Pennsylvania German stock. His immediate forebears had never moved from this location since his great-grandfather had settled there in 1751, having emigrated from Getzigen in Wurtemburg, Germany.

Mr. Brendlinger prepared for a college education in the country schools of his home district near Pottstown, Pa., and at the Tremont Academy, at Norristown, Pa. He was graduated as a Civil Engineer in June, 1870, from the Polytechnic College of the State of Pennsylvania, in Philadelphia.

From August, 1870, to August, 1871, he was an Assistant Engineer on the location and construction of the St. Louis, Council Bluffs, and Omaha Railroad, from St. Louis, Mo., to Omaha, Nebr. Between October, 1871, and November, 1874, he served as Assistant Engineer on the Pittsburgh, Virginia, and Charleston Railroad, from Pittsburgh, up the Monongahela River, to Brownsville, Pa.; and in March and April, 1875, he was in charge of the Right-of-Way Department of the Allegheny Valley Railroad Company.

From April, 1875, to June, 1877, Mr. Brendlinger was Chief Engineer of Construction of the Point Bridge across the Monongahela River at Pittsburgh, surveys for which were started in 1875. The City Engineer of Allegheny, Pa., was the Consulting Engineer, and Mr. Brendlinger was Engineer in charge of the work, while the late Samuel Rea, Hon. M. Am. Soc. C. E., was an

* Memoir prepared by Clark Dillenbeck and Charles S. Churchill, Members, Am. Soc. C. E.

Assistant Engineer. The contract was awarded to the American Bridge Company, of Chicago, Ill. At that time the Point Bridge was the longest span chain bridge in the world. Designed by Edward Hemberle, an Austrian by birth, it had a span of 800 ft. In 1904, it was generally rebuilt and, in 1921, was replaced by a heavier and more modern structure.

From June until July 4, 1877, Mr. Brendlinger was in charge of testing American iron and steel for the United States Commission, headed by the late Q. A. Gillmore and Alexander L. Holley, Members, Am. Soc. C. E., and W. Leroy Smith. Between July 4, 1877, and November, 1878, he served as Assistant Engineer on the location of the Pittsburgh and Lake Erie Railroad from Pittsburgh to Youngstown, Ohio; later, he became Resident Engineer of Construction in charge of the Ohio River Division.

From November, 1878, to July, 1881, Mr. Brendlinger was Chief Civil Engineer for the Edgar Thompson Steel Works, at Braddock, Pa., which work included surveys locating iron mines in Centre County, with headquarters at Bellefonte, Pa. He resigned from the Steel Company in 1881 to become Assistant Chief Engineer of an important railroad project. In this connection, he was the recipient of a short autographed letter from Andrew Carnegie, reading in part as follows:

"Cresson, Sept. 21, 1881

"MY DEAR MR. BRENDLINGER,

"Yours duly rec'd. It is always painful to me when one of the corps leaves the service but I can scarcely wonder that you longed to get back to Railroading.

"That is a great line you are building—a great property—I hope it will give Pittsburgh fair rates upon Coke. * * *

"With best wishes for your success,

"I am truly yours,

"ANDREW CARNEGIE."

From July 1, 1881, to January, 1884, Mr. Brendlinger was Assistant Chief Engineer for the Pittsburgh, McKeesport, and Youghoigheny Railroad during its location and construction from Pittsburgh to Connellsville, Pa., a distance of 60 miles. Between 1878 and 1881, the mileage of railroad construction in the United States was comparatively small, due to heavy mortgages and many receiverships on existing railroads. This undertaking in 1881 was the first of considerable moment in Western Pennsylvania. It was planned to be constructed on a relatively high standard as to roadbed, masonry, bridges, and track; and Mr. Brendlinger was selected to carry out these plans. He took great care in selecting the entire Engineering Department and energetically followed up every detail necessary to bring about the desired results. The line was opened for traffic on November 19, 1883, at a cost of about \$5 000 000 and was in such good condition that, during 1884, it earned 6% interest on its cost, together with a surplus, and thereafter continued this good record.

From January 1, 1884, to April, 1888, Mr. Brendlinger was Chief Engineer of the Pennsylvania, Schuylkill Valley Railroad Company and of all Pennsylvania Railroad Lines between Hamburg and Nescopeck, Pa., on what is

now the Sunbury Division of the Pennsylvania Railroad, on the North Branch of the Susquehanna River. Surveys were started on January 15, 1884, on that part of the line north of Hamburg, covering a section where there were difficult questions to be settled between the new line and both the Reading Railroad and the Schuylkill Canal; also, difficult questions arose from the necessity of crossing Broad Mountain. Many miles of preliminary lines were run, and topographical maps made, covering a large area between Hamburg and Hazleton, Pa., before the best route could be determined.

The final location, which included an approximately right-angle crossing of the outcrop of the Mamouth Coal Measure of the Mahanoy Valley, within the Stephen Girard Estate, where the coal has a thickness of 40 ft., involved the purchase of a pillar of coal as a support of the railroad with a depth of about 300 ft. The line was located in the spring of 1885 and contracts were awarded by Mr. Brendlinger in April of that year for a railroad with roadbed, masonry bridges, and track in accord with the standards of the Pennsylvania Railroad Company. The same characteristic expedition which he had practiced in Pittsburgh was used by Mr. Brendlinger, with the result that the line was completed and operated between Philadelphia and Pottsville, Pa., in November, 1886; and through to Nescopeck on the Pennsylvania Railroad, and thence, to Wilkes-Barre, Pa., in April, 1887, the new lines constructed north of Hamburg, covering 71 miles. Mr. Brendlinger received great credit for the completeness of the construction records and plans which were acclaimed as a great aid to the Operating Department of the Pennsylvania Railroad Company.

From April, 1888, to April, 1892, Mr. Brendlinger was General Superintendent of Construction for Brown, Howard, and Company of New York, N. Y., on the New Croton Aqueduct, with headquarters at Tarrytown, N. Y.

In 1892 he organized the firm of Brendlinger and Nearing, which was engaged in general contracting, doing work, until 1896, for the City of Yonkers, N. Y., for the Buffalo, Rochester and Pittsburgh Railroad Company, and for the Long Island Railroad Company. He also was associated with Pennell and O'Hearn, Contractors, on railroad construction for the Pennsylvania Railroad Company.

From 1896, until he retired from active business in 1914, Mr. Brendlinger was a General Contractor engaged on work for the Pennsylvania Railroad Company, the Pittsburgh, Cincinnati, Chicago, and St. Louis Railway Company, and the Philadelphia and Reading Railway Company, completing the following works: For the Pennsylvania Railroad Company, change of line, Pomeroy, Pa., and Oxford, Pa., at Milbourne Mills, League Island, and Lewistown Narrows, and on the Gallitzin Tunnels; for the Pittsburgh and Erie Railroad Company, Renovo to Keating, Pa.; for the Susquehanna Coal Company, at Millersburg, Pa.; for the Philadelphia and Reading Railroad Company, at Bowmansdale, Pa., Shippensburg, Pa., Barnitz, Pa., and Hopewell, N. J.; for the Baltimore and Ohio Railroad, the Baltimore and Ohio Southwest Yard, at Philadelphia; for the Cleveland Short Line Railroad Company, at Cleveland, Ohio; and for the Pittsburgh, Cincinnati, Chicago, and St. Louis Railroad at Cambridge City, Ind.

Mr. Brendlinger was an easily approached, helpful man and a good friend. He exercised an uplifting influence on those with whom he was associated. His thoroughness and care in his engineering practice was, after 1888, followed in his contract work. The high standards of his entire life work were followed in all business and social dealings with others.

He died in Philadelphia, Pa., on December 16, 1929, from heart failure, caused by shock and exhaustion resulting from a fractured hip, due to a fall five weeks previous. He was within a month of completing his eightieth year, and prior to the accident had retained the keen and alert mental faculties which had always been his.

He was married to Hannah Emily Brown, on October 24, 1872, who died on April 27, 1924. There survive him four children: Margaret R. Brendlinger, George F. Brendlinger, William B. Brendlinger, and Mary B. Olmsted.

Mr. Brendlinger was elected a Member of the American Society of Civil Engineers on September 7, 1887.

SAMUEL MORSE FELTON, M. Am. Soc. C. E.*

Died March 11, 1930.

Samuel Morse Felton was born in Philadelphia, Pa., on February 3, 1853. He came of old English stock, his ancestors having settled in Salem, Mass., in 1633. His earliest paternal ancestor was Nathaniel Felton, Ensign and Lieutenant in the Salem Foot Company, who came to America from Great Yarmouth, Norfolk, England. His father, Samuel Morse Felton, was Superintendent of the Fitchburg Railroad in 1843, and, in 1861, as President of the Philadelphia, Wilmington and Baltimore Railroad Company, arranged for the secret passage of Abraham Lincoln from Harrisburg, Pa., to Washington, D. C., at the time of his first inauguration. He afterward became President of the Pennsylvania Steel Company. His mother, Maria Low (Lippitt) Felton, was the seventh in descent of Roger Williams, Captain in King Philip's War and Governor of Rhode Island Colony, in 1654.

Mr. Felton's inclination and early training were along railroad lines. He was educated in private schools in Philadelphia, and, for two years, took an engineering course at the Pennsylvania Military Academy, at Chester, Pa., working during vacations as Rodman on the Chester Creek Railroad. After leaving the Academy at Chester he spent one year in the field with a locating party on the Lancaster Railroad, in New England, as Leveler and Assistant Engineer. Realizing the value of further technical training he entered the Massachusetts Institute of Technology, at Boston, Mass., and was graduated in 1873 with the degree of Civil Engineer. After his graduation, because of his earlier experience in the field, he was able to accept a position as Chief Engineer of the Chester and Delaware Railroad, a small line being constructed by the Reading Railroad Company along the Delaware River.

His ambition was to become the operating head of a railroad system, and, in 1874, at the age of twenty-one, he was appointed General Superintendent

* Memoir prepared by W. G. Lerch, Vice-Pres., C. G. W. R. R., Chicago, Ill.

of the Pittsburgh, Cincinnati, and St. Louis Railroad Company, with which he remained until 1882. In 1881, Mr. Felton was also made General Superintendent of the Little Miami, and Cincinnati and Muskingum Valley Railroad Companies. In 1882, he became General Manager of the New York and New England Railroad Company; in 1884, Assistant to the President of the New York, Lake Erie, and Western Railroad Company, and General Manager of the New York, Pennsylvania, and Ohio Railroad Company, both of which are now a part of the Erie System. In 1885, he was made Vice-President in Charge of Traffic, later taking over, in addition, the duties of Vice-President in Charge of Operation. He remained in these positions until 1890, when he was elected President of the East Tennessee, Virginia and Georgia Railway Company and, later, President of the Alabama Great Southern Railroad and the Cincinnati, New Orleans, and Texas Pacific Railway Companies. In 1893, when the latter company went into a receivership, Mr. Felton was appointed Receiver of that property by former President William H. Taft, who, at that time, was on the Federal Bench in Cincinnati, Ohio. He also in that year became Receiver of the Columbus, Sandusky, and Hocking Railway Company and the Kentucky and Indiana Bridge Company. He directed the operations of these properties until 1899, when the late E. H. Harriman secured control of the Chicago and Alton Railroad and, finding complete rehabilitation necessary, decided that Mr. Felton was best fitted to undertake the work. He prevailed upon him to come to Chicago, Ill., as President of the road, where he remained until 1907. During this time he rebuilt much of the line, installed modern facilities and equipment, and the Alton Railroad, when he left it, was as well equipped as any road in its territory. In December, 1907, he was elected President of the Mexican Central Railway Company and the Mexican American Steamship Company, continuing in those positions until the Mexican Government merged all the principal railroads in the Republic into the National Railways of Mexico.

In 1909, Mr. Felton returned to the United States and became Chairman of the Tennessee Central Railroad Company. In the fall of that year after making an examination and report on the road for the re-organization managers he was elected President of the Chicago Great Western Railroad Company and its affiliated lines. He held this office until the end of 1925, when he was made Chairman of the Board of Directors, in which position he remained until his death. From 1912 to 1914 he was also Co-Receiver and President of the Pere Marquette Railway Company. When Mr. Felton took charge of the Chicago Great Western Railroad Company, it had just emerged from a receivership, but within a period of six years he completely rehabilitated the road and its equipment and placed the property on a dividend-paying basis. During the period of Federal operation of railroads incident to the World War, the road was not maintained, and when it was turned back to its owners in 1920, it was in a bad state of repair with its traffic diverted to other roads, necessitating doing over again what Mr. Felton had begun in 1910—the reconstruction and rehabilitation of the road and its equipment and the task of getting back its traffic. He went vigorously ahead and had practically completed the work with prospects for renewed prosperity of the

Company in sight when illness assailed him, and he was compelled to relinquish the active direction of the railroad to others.

Mr. Felton maintained his interest in railroad problems, and was a member of the Executive Committee of the Association of Railway Executives, of the Western Association of Railway Executives, and of the Western Railway Committee on Public Relations. He was also a Director of the American Railway Association and President of the Western Railroad Association. During his railroad career he served on several important engineering commissions, having acted as Chairman of the Engineering Commission of the City of Cincinnati to select the location for a new water-works, and, in 1900, he was engaged by the Commercial Club of St. Louis, Mo., to report on a municipal bridge project. At different times, he made reports for banking interests and re-organization managers on 23 railroads aggregating more than 32 000 miles.

Mr. Felton's most spectacular service was probably in connection with the Spanish-American and the World Wars. During the former, while in charge of the Cincinnati, New Orleans and Texas Pacific Railway, his road moved a larger number of troops than any other single line. At the outbreak of the war it handled the first troop movement south to Chattanooga, Tenn., which consisted of 31 trains leaving Cincinnati 10 min. apart and reaching Chattanooga on time, without delay or accident; and this was accomplished over a single-track road.

In June, 1916, while acting as President of the Chicago Great Western Railroad, he was requested by the War Department to organize troops to operate the railroads in Mexico in the event the United States Army found it necessary to advance south of the border. His title was Consulting Engineer and Adviser to the Chief of Engineers, United States Army. He went zealously to work and, in a short time, engaged a sufficient number of railroad men, familiar with Mexican railways, to fill the most important positions thereon, and arranged under option, if needed, for locomotives, construction and wrecking trains, rails, and cross-ties to replace destroyed track, telephone and telegraph equipment, and other material. In June of that year he joined the Staff of General Funston, at San Antonio, Tex., and reported sufficient men and material ready for the initial movement of troops. This, however, became unnecessary as the tension between the two countries relaxed.

In the meantime the question of forming a reserve force of railroad men in the Engineer Corps was considered by the War Department, and Mr. Felton was requested to plan its organization. While this work was progressing, the United States entered the World War and the original plan was enlarged upon, as it was found necessary to organize, at once, nine regiments. The work of equipping and drilling these regiments was hurried, and they were among the first American troops to land in France. Mr. Felton was appointed Director General of Railways, United States Army, the title later being changed to Director General of Military Railways. He took up his residence in Washington and went actively to work organizing additional regiments, being empowered by the Secretary of War to make contracts and purchases, at his own discretion, without going through the customary for-

malities. Two days after his appointment as Director General of Railways, he contracted for the building of three hundred locomotives and in twenty working days thereafter the first one was crated ready for shipment. The total amount of contracts made under his direction was $612 000 000, covering: 3 750 standard gauge locomotives; 1 547 narrow-gauge locomotives; 91 519 standard-gauge cars; 8 930 narrow-gauge cars; 823 locomotive cranes; 76 gantry cranes; and materials for more than 100 water stations, 749 345 tons of rail (the equivalent of 5 951 miles of single track), and other items of like proportion. By careful buying a saving of more than $33 000 000 was made in connection with these contracts.

In addition to the purchasing of materials and supplies, 84 000 transportation men were organized in battalions and regiments for service overseas and, at the time of the Armistice, 70 000 transportation men were in France and 14 000 more ready to sail.

In 1918 Mr. Felton made a trip of inspection to France, traveling among all the battle lines. On May 23 of that year, he was made Vice-President of the Port and Harbor Facilities Commission of the United States Shipping Board, and, in 1918, became Acting Chairman, performing the duties of the office until he left Washington. He resigned as General Director of Military Railways on December 1, 1918.

Mr. Felton emerged from the war without military rank. He was twice offered a commission as Brigadier General, but refused, believing that a place in the military organization would prove a hindrance rather than a help in his work. He was awarded the Distinguished Service Medal of the United States Army, the first one granted to a civilian. In further recognition of his war work, the French Government made him an Officer of the Legion of Honor.

After the close of the war he was appointed Honorary Adviser to the Army Industrial War College, a member of the War Department Business Council, and Chairman of the Committee on Military Affairs of the Association of Railway Executives.

Notwithstanding the heavy responsibilities imposed upon him by his railroad duties, Mr. Felton found time to interest himself in many outside mercantile and financial activities, the success of which bore witness to his ability, and added luster to the corporations with which he was identified. He was a Director of the Central Trust Company of Illinois, the Central Securities Company, the Peoples Trust and Savings Bank of Chicago, the Equitable Life Assurance Society, the Chicago Surface Lines, and the Colonial Land Company. He was a member of the Western Society of Engineers, American Railway Engineering Association, Franklin Institute, American Legion, Civil Legion, Society of American Military Engineers, Veterans of Foreign Wars, Ohio Society, Sons of the Revolution, Ohio Society of Colonial Wars, and the New England Society, in New York; the Commercial Club of Chicago, Chicago Club, Chicago Athletic Association, Saddle and Cycle, and Old Elm Golf Club, of Chicago; the University Club of New York and the Commercial Club of Cincinnati.

There was conferred upon him the degree of Doctor of Laws by the Pennsylvania Military Academy and by Marietta College, of Marietta, Ohio. He was a member of the Corporation of the Massachusetts Institute of Technology.

Mr. Felton was married in 1880, to Dorathea Hamilton, of Pittsburgh, Pa. Mrs. Felton died in 1923. They had three daughters and one son.

Mr. Felton was elected a Member of the American Society of Civil Engineers on January 4, 1882.

GEORGE CHARLES KREUTZER, M. Am. Soc. C. E.*

Died November 23, 1929.

George Charles Kreutzer was born on a ranch in Douglas County, Colorado, on March 6, 1884. He was the son of Edward D. and Jane Eliza (Keppel) Kreutzer. His father came to America from Germany in 1867, remained in New York and Pennsylvania for a few years, and then, in 1874, with his wife, a native of England whom he had married the preceding year, moved to Colorado where he finally established himself on a ranch in Douglas County about ten miles west of Sedalia.

Mr. Kreutzer's boyhood days were spent in hard work on his father's ranch where he gained that background of practical experience in agriculture, irrigation, and lumbering, which was to prove so valuable to him in later years.

He attended Grammar School at Sedalia and High School in Castle Rock, Colo. He entered the Colorado Agricultural College at Fort Collins, Colo., in 1903 and was graduated in 1908, with the degree of Bachelor of Science in Engineering, having taken his major work in civil and irrigation engineering. He worked his way through college under great difficulties, but even so, he made an excellent scholastic record and was an outstanding leader in all student activities. In 1922, the degree of Master of Science was conferred upon him by his Alma Mater.

On leaving college in 1908, Mr. Kreutzer's first employment was as Special Field Agent, Bureau of Irrigation Investigation, United States Department of Agriculture. Although he only occupied this position for a few months, it was the beginning of a useful career in the public service, both State and National, largely in irrigation development and land settlement problems.

From September, 1908, to April, 1910, he was employed by the Colorado Experiment Station, and was engaged in the studies of practical irrigation problems, animal husbandry, and agronomy. In April, 1910, Mr. Kreutzer again entered the employ of the United States Government for a brief period, this time with the Bureau of Reclamation, Department of the Interior, as Assistant Superintendent of Irrigation, on the Shoshone Project, in Wyoming, where he supervised the operation of the Garland Division during the summer and had charge of all survey parties during the fall and winter. He continued in this position until January, 1911.

* Memoir prepared by Elwood Mead and R. F. Walter, Members, Am. Soc. C. E.

In 1910, Elwood Mead, M. Am. Soc. C. E., Chairman of the State Rivers and Water Supply Commission of the State of Victoria, Australia, was visiting the United States. Among its other duties, this Commission operated all irrigation and water supply works serving the farming districts. While in the United States Dr. Mead asked Dr. Charles A. Lory, President of the Colorado Agricultural College, to suggest some one qualified to serve as Superintendent of one of the Australian projects, the requirements for the position being marked executive ability combined with good engineering and agricultural training and experience. Mr. Kreutzer was recommended by Dr. Lory and, as a result, he accepted an offer made by the Commission and early in 1911 was appointed Superintendent of the Cohuna Irrigation District in Northern Victoria, Australia. This assignment, which first brought Mr. Kreutzer in contact with Dr. Mead, was the beginning of an enduring friendship and professional relationship. Mr. Kreutzer's success in this work was so marked, and the progress of the District under his administration so satisfactory, that he was later transferred to the Central Office in Melbourne, Victoria, Australia, and made a general Economic Adviser in the creation of new projects and the reconstruction of old ones. He continued in this work until 1916 when, owing to the interruption in irrigation development in Australia, due to the World War, he resigned his position and returned to the United States.

In July, 1916, Mr. Kreutzer was appointed Agricultural Adviser of Kern County, California, as a part of the Agricultural Extension Service of the University of California, in co-operation with the U. S. Department of Agriculture. In this work he gave special attention to co-operative marketing and war production. His duties were later enlarged to include the supervision of county agents engaged in similar work in Kern, Kings, Tulare, and Fresno Counties. In 1918, the Legislature of California passed the State Land Settlement Act, to be under the control of a commission of which Dr. Mead was Chairman, he having returned to the United States in advance of Mr. Kreutzer. This Act had for its purpose the creation of planned farming communities of home owners organized to co-operate and to have the benefit of long-time payments and low interest rates in the purchase and improvement of their farms. Mr. Kreutzer was made Superintendent of the Durham Settlement, the first to be established under the Act, and remained in this position, which he filled with great fidelity, until 1924.

Continuing a professional association of many years' standing, Mr. Kreutzer followed when Dr. Mead was appointed by the President, in the spring of 1924, as Commissioner of Reclamation, U. S. Department of the Interior. Concordantly, in June of that year Mr. Kreutzer was appointed Chief of the Division of Reclamation Economics, under Dr. Mead. In this position, which he held until his death, he was untiring in his efforts to improve the economic status of the settlers on the Federal Irrigation projects.

He was married on March 4, 1914, in Melbourne, Australia, to Dorothy McFarlane, the daughter of an English clergyman. They had two children, William Elwood and Adelaide Dorothy, both of whom, with his widow, survive

him. Also surviving him are one brother, Mr. William R. Kreutzer, and three sisters, Mrs. Minnie Cell, Mrs. Bessie Grandy, and Mrs. Agnes Klant.

Mr. Kreutzer brought to all his work the influence of a remarkable personality. In it were combined warm human sympathy, and a delightful sense of humor, together with unusual clear business sense. He was a man of much natural ability, fine personality, and extraordinary skill in dealing with people of all walks of life. He was honest, gracious, frank, and generous, loved by all who knew him for his sterling qualities of character, and respected and admired by all his co-workers for his outstanding ability and fairness.

Mr. Kreutzer was elected a Member of the American Society of Civil Engineers on January 16, 1928.

WILLIAM EDWARD McCLINTOCK, M. Am. Soc. C. E.*

Died March 2, 1930.

William Edward McClintock was born in Hallowell, Me., on July 29, 1848, the son of Captain John McClintock, a well known navigator, familiar with every sea, who crossed the Pacific with an ordinary watch for a chronometer and a school atlas for a chart, and Mary Bailey (Shaw) McClintock. On his paternal side he was of Scotch-Irish ancestry, descended from William McClintock, a defender in the Siege of Londonderry in 1689. His earliest American ancestor was William McClintock, who came to the United States, a babe in arms and whose parents settled in Medford, Mass., in 1735. On his mother's side, he was descended from the Rev. John Bailey, a prominent Puritan clergyman.

Mr. McClintock's early education was acquired in the public schools, supplemented by a four-year course at Hallowell Academy, and one year at Kent's Hill Seminary, at Readfield, Me. His early manifested taste for engineering was inherited from his paternal family, his grandfather, William McClintock, having been an expert land surveyor, some fine examples of whose work are now on file in the State of Maine.

On the completion of his preparatory education Mr. McClintock entered a period of training for his profession, including both office and field work, and he also received instruction from a private tutor. His first work as an engineer was in connection with the United States Coast and Geodetic Survey with which he was engaged from 1867 to 1876 on work in Maine, Massachusetts, New York, North Carolina, South Carolina, Georgia, Florida, Louisiana, and Mississippi. From 1876 to 1879 he was employed on a survey of the City of Portland, Me., and from 1877 to 1879 on a survey of Boston Harbor, and also on a re-location survey for the Boston and Maine Railroad Company, including all its branches in Massachusetts. In 1880, he was elected City Engineer of Chelsea, Mass., which position he retained until 1890 when he went into business with Mr. J. Leslie Woodfall under the name of McClintock and Woodfall.

* Memoir prepared from data on file at Society Headquarters.

Mr. McClintock's special engineering works have included surveys for the South Pass Jetties, at the mouth of the Mississippi River; surveys for the improvement of the harbors of New York, N. Y., Boston Mass., and Portland, Me.; surveys for the improvement of the Saco, Savannah, Pamlico, St. Mary, St. John, and Nassau Rivers; and the installation of sewerage systems for the cities and towns of Chelsea, Revere, Gardner, Westfield, East Hampton, Andover, Lenox, Natick, and Lexington, Mass.; Bennington, Vt.; Bath and Calais, Me.; Exeter, N. H.; and St. Stephen and Milltown, N. B., Canada. He also served as Consulting Engineer on sewer and water-works construction in Holyoke, Spencer, North Brookfield, North Attleboro, and several other towns in Massachusetts as well as in other States.

Mr. McClintock was actively identified with the Good Roads Movement in Massachusetts and, in its advocacy, he wrote various papers, and delivered addresses in nearly every city and town in the Commonwealth. In 1892 the old League of American Wheelmen, which had been agitating the question of better roads for several years, was publishing a magazine in an effort to stimulate public interest. At that time Mr. McClintock was building roads in various towns in Massachusetts, Maine, and New Hampshire. Through the efforts of the League, Governor W. E. Russell appointed a Commission to investigate and report on the improvement of highways in Massachusetts, and Mr. McClintock was appointed as the Engineer Member, together with George A. Perkins, a lawyer, and Nathaniel Shaler, widely known as Professor of Geology at Harvard University. The result of the studies of this Commission showed that outside the thickly settled communities there were practically no good roads in the State.

In 1893, the Legislature passed an Act providing for the establishment of the Massachusetts Highway Commission and these same three men were appointed to form the new Commission. Mr. McClintock was subsequently re-appointed by Governors Greenhalge, Walcott, Crane, Guild, and Bates. In 1898, he was made Chairman of the Commission which position he retained until 1908 when he relinquished this work to give his services to his own home town which had been recently devastated by fire. He may be identified as a pioneer in the Good Roads Movement, and had much to do with organizing the Engineering Staff that was to carry out the Massachusetts system of road-building, a system which has been widely studied and copied throughout the country. He was nationally known as the "Father of Good Roads."

It was during his administration as Chairman of the Massachusetts Highway Commission that the increasing motor-vehicle traffic showed the impracticability of the water-bound macadam highway which until that time had been considered the finest road in existence. Under Mr. McClintock's leadership the bituminous macadam highway was devised, and it has since been the standard for the superior motor highway, except in sections of the heaviest travel where concrete has been adopted.

In 1908, came the Chelsea fire, during which more than 17 000 people were rendered homeless, and churches, public buildings, and schools were destroyed. Mr. McClintock was the man to whom every one looked to take charge of the

relief operations, and so effectively was this done that when it was decided that Chelsea was to have an emergency commission form of government for five years, Mr. McClintock was selected by Governor Eben Draper as Chairman of the Board of Control. In this capacity he again had an opportunity to show his executive and constructive ability in the rebuilding of the city, and to this end he gave unremittingly of his time and service. Despite some opposition and many difficulties, this Board brought Chelsea out of its stricken condition. It rebuilt all the public buildings, laid out new streets, and left the city with better structures than it had previous to the fire. It was while he was engaged whole-heartedly in this work that he was recognized by Governor Charles E. Hughes, of New York State, as the leading authority in the United States on the construction of modern highways, and he was invited to accept the Chairmanship of the New York Highway Commission. He refused this offer because he was devoted to his community and to the work on hand.

In 1913 when the city returned to the Mayor and Board of Aldermen form of government, he was appointed Superintendent of Chelsea Ferry which position he held until the ferry was discontinued. He then became connected with the State Department of Education, in its field of University Extension. He died at his home in Chelsea on March 2, 1930, in his eighty-second year.

Mr. McClintock served on the Board of Aldermen, and School Committee, and was a Vice-President of the Chelsea Savings Bank. He was one of the organizers and first President of the Massachusetts Highway Association, a member of the Boston Society of Civil Engineers, and, in addition, an Instructor in Highway Engineering at Lawrence Scientific School, at Harvard University. He was a member of the Robert Lash Lodge of Masons, and the Royal Arch Chapter of the Shekinah, and of the Review Club. He was prominently connected with the Church of the Redeemer (Universalist) of Chelsea.

He was married in Portland, Me., in 1873, to Mary Estella Currier. They had five children all of whom are living: William J., of Melrose, Mass., Francis B. and Samuel, of Chelsea, Paul, of Hillside, N. J., and Dorothy, of Chelsea.

Mr. McClintock was elected a Member of the American Society of Civil Engineers on December 5, 1888.

GUY MOULTON, M. Am. Soc. C. E.*

Died December 3, 1929.

Guy Moulton, the son of Emery and Mary J. (Churchill) Moulton, was born on February 25, 1861, in Cicero, N. Y.

He received his early education in Cicero, later enrolling at Cornell University, in Ithaca, N. Y., from which institution he received the degree of Bachelor of Science in 1881. Following his graduation, Mr. Moulton was employed by the Buffalo, Rochester and Pittsburgh Railroad Company, serving

* Memoir prepared by Glenn D. Holmes, M. Am. Soc. C. E.

as Stakeman, Draftsman, Transitman, and Leveler until the autumn of 1883. In the early summer of that year, he held the position of Assistant Engineer on location for the Pittsburgh and Allegheny Railroad.

From the autumn of 1883 to the spring of 1889, Mr. Moulton was engaged in other business, but returned to the Engineering Profession when he accepted the position of Transitman for the Lizard Creek Branch of the Lehigh Valley Railroad. In July, 1889, he was promoted to be Assistant Engineer in charge of construction on the Buffalo Extension under the late Paul S. King, M. Am. Soc. C. E., Chief Engineer.

Mr. Moulton left the service of the Lehigh Valley Railroad Company in December, 1892, to design and supervise the construction of an addition to the sewerage system at Watkins, N. Y., which work was successfully completed in the spring of 1893. Following a brief engagement with the New York and New England Railroad Company as Assistant Engineer, a similar position was accepted with a survey party for a private railroad in Tennessee during the summer of 1894. During the succeeding year he acted as Superintendent and General Manager for Charles McFadden in mining bituminous coal in Pennsylvania.

During the winter and spring of 1895-96 Mr. Moulton served as Assistant Engineer in charge of construction of the Jackson and Saginaw Railroad, in Michigan. In September, 1896, he assumed charge as Engineer and General Manager for McDonald and Sayre, General Contractors on construction work of the Erie Canal Improvement. In November of the following year, he resigned to accept an appointment as First Assistant Engineer for the State of New York. He was in charge of the work from Syracuse, N. Y., to the Oneida County line during the "$9 000 000 Improvement" and after its completion, he was given charge of the entire Middle Division.

In 1904, as Resident Engineer, Mr. Moulton had charge of Residences 6 and 7, and a part of Residence 5, of the Barge Canal System. In 1909, he was appointed Division Engineer and, in 1911, First Resident Engineer of the System. In 1915 he was again made Division Engineer of the Barge Canal which position he held until his retirement in 1922.

From 1923 until his death he was engaged in consulting practice, maintaining an office in Syracuse. Mr. Moulton had been in failing health following a major operation in the summer of 1929, but continued his usual activity to the end. Death came to him as he was leaving his son's automobile on the way to his office, the result of a heart attack.

Mr. Moulton was known among the members of the Engineering Profession as a capable engineer of scrupulous integrity; to the public, as a trustworthy servant; among his associates, as a constant and unchanging friend; and to his family, as a devoted and generous husband and father.

He was a member of the First Universalist Church, which he served in many ways. He was affiliated with the Masonic bodies and was a member of the Citizens' Club of Syracuse. As an active member of the Syracuse Technology Club, he served as Director, Vice-President, and President, and on many of its committees. Mr. Moulton ably served his city upon two Flood

Prevention Committees when appointed in 1915 and 1925 to study flood hazards and remedies in Syracuse.

He was a great lover of Nature. During the early morning and evening hours he took especial delight in his garden and the growing of roses, dahlias, gladioli, and other flowers.

He was married on March 16, 1887, to Sarah Adeline Wright, of Clay, N. Y., who, with three sons, Webster C., Lloyd W., and Guy W. Moulton, survives him.

Mr. Moulton was elected a Member of the American Society of Civil Engineers on March 1, 1905.

RALPH WALDO NICKERSON, M. Am. Soc. C. E.*

Died May 13, 1929.

Ralph Waldo Nickerson, of an old New England family, was born on November 26, 1881, at Tiverton, R. I. His father was Capt. George Ferdinand Nickerson, and his mother, Marie (Springer) Nickerson, was a lineal descendant of Knight Springer of Revolutionary Service.

Mr. Nickerson prepared for college in the public schools of Tiverton. He entered Worcester Polytechnic Institute, Worcester, Mass., in the fall of 1899 and was graduated in 1903 with the degree of Bachelor of Science. Immediately after his graduation he entered the employ of the American Bridge Company as a Structural Draftsman in the East Berlin, Conn., Plant. He was promoted to the position of Squad Leader in 1906, and had supervision of a squad of draftsmen preparing detail drawings of bridges and buildings. In 1911 he was again promoted, this time to the position of Designing Engineer in the New York Office of the Company.

As a Detail Draftsman he had charge of the detailing of some of the most important structures fabricated by the Company, and as a Designer he designed the steel framework for many large manufacturing plants, among which were the new plants of the American Brass Company at Waterbury, Ansonia, and Torrington, Conn., and Kenosha, Wis. He also had general supervision of the detailing of his own designs.

During his connection with the American Bridge Company Mr. Nickerson was released to do special work for H. G. Balcom, M. Am. Soc. C. E., Consulting Engineer of New York City, under whose direction he prepared designs for several large office buildings there.

In December, 1924, he resigned his position with the American Bridge Company and opened an office in New York City, as Consulting Engineer, making a specialty of structural steel, reinforced concrete, and foundations. While in private practice Mr. Nickerson designed the structural steelwork and foundations for a large number of theatres, office buildings, and miscellaneous structures, in various parts of the country, and established a reputation for conscientiously thorough and high-class work.

* Memoir prepared by J. E. Wadsworth, J. B. French and H. G. Balcom, Members, Am. Soc. C. E.

In 1928 Mr. Nickerson entered the Engineering Department of the George A. Fuller Company and remained with that Company until his death on May 13, 1929. During this period he had the direct supervision of the work of the Company on the No. 400 Madison Avenue Building, the Beaux Arts Apartments (312 and 313 East 44th Street), and the Astor Apartment (520 East 86th Street), all in New York City.

Mr. Nickerson was a man of sterling character and of indisputable integrity. He was an indefatigable worker and a man of unquestioned reliability. He had the confidence of the companies by which he was employed and of his clients for whom he acted as a Consultant.

He was a member of the Union County (New Jersey) Chapter of the Society of Professional Engineers and Land Surveyors, the Sons of the American Revolution, Elizabethtown, Pa., Chapter; and the Architects and Engineers Square Club. He was also a member of Hillside (N. J.) Lodge No. 241, F. and A. M., and an Elder in the Hillside Presbyterian Church.

He is survived by his widow, Edith North Nickerson, and two sons, Ralph Waldo and Stanley North Nickerson.

Mr. Nickerson was elected a Member of the American Society of Civil Engineers on April 20, 1925.

GEORGE HARVEY NORTON, M. Am. Soc. C. E.*

Died March 4, 1930.

George Harvey Norton, the son of George Harvey Norton, M. D., and Martha Graves Norton, was born in East Pembroke, N. Y., on October 24, 1863. In 1887, he was graduated from Cornell University, at Ithaca, N. Y., with the degree of Civil Engineer.

In his early life, during 1887 and 1888, Mr. Norton was engaged in survey work on the Duluth, South Shore, and Atlantic Railroad and the Chicago and West Michigan Railroad. While thus engaged, he went to Buffalo, N. Y., on a vacation and obtained a position as Substitute Draftsman with the Grade Crossing Commission. Within six months, he was appointed Assistant Engineer, under the late George E. Mann, M. Am. Soc. C. E., who for years served both as Engineer for the City of Buffalo and the Grade Crossing Commission. Mr. Norton entered the service of the City of Buffalo on November 1, 1888, and was closely associated with all its more important engineering projects from that time until his death.

Among his notable accomplishments, were the improvement of Scajaquada Creek and, coincident with this, the improvement of the Buffalo River and Cazenovia Creek, for relief from floods. From 1889 to 1895, as Assistant Engineer, he also constructed and operated a temporary sewage pumping plant, made paving plans, and had charge of the underground construction preceding paving, on about 150 miles of streets. During 1896 and 1897, he was engaged in paving construction.

From 1898 to 1908, Mr. Norton, as Assistant City Engineer, was in immediate charge of all bridge, harbor, and hydraulic work of the city (except

* Memoir prepared by Edward P. Lupfer, M. Am. Soc. C. E.

grade-crossing elimination), covering both construction and maintenance, including the maintenance of many grade-crossing structures after completion. He was in charge of the design and construction of several fixed bridges, having spans of from 40 to 150 ft., and the general location, design, and construction of temporary structures and foundations for two bascule bridges, one of 110-ft. span and the other of 166-ft. span, together with the supervision of the construction and operation of these bridges.

His name is likewise outstanding in the development of Buffalo's waterfront. He had immediate supervision of the plans and the dredging of Buffalo's Inner Harbor, this project costing approximately \$900 000, and, during 1908, he was engaged in survey and engineering work in negotiations for the improvement of a large part of Buffalo's Outer Harbor, as well as in making hydraulic studies and plans for flood abatement and extending the navigable channel in Buffalo River, at a total cost of about \$1 000 000.

From 1909 to 1923, as City Engineer, Mr. Norton was in charge of all street, sewer, sidewalk, and bridge construction and maintenance, all city harbor deepening and extension, land surveys, the supervision of franchise, occupation of streets, and valuation of physical properties therein, as well as a great variety of other engineering work arising in a large city. Some of the larger projects included the improvement and extension of navigation; the beginning of the Scajaquada Creek drain, one of the largest of such projects for placing streams below ground; water-front improvements; studies of sewage treatment; and the settlement of long disputed water-front rights and titles. Expenditures in this work, under the City Engineer, averaged about \$3 000 000 per year.

As City Engineer, Mr. Norton also was made Chairman of the City Planning Committee, and served as such until the spring of 1926. During this time there was developed a general City Plan, including the location of public buildings and the adoption of a zoning ordinance. He was also a Director of the Buffalo City Planning Association and Niagara Frontier Association, and was deeply interested in general city planning.

In 1923, Mr. Norton became Chief Engineer of the Grade Crossing and Terminal Station Commission, and served in this capacity until his death. His career in this office was a notable one, and as Chief Engineer of the Grade Crossing Commission, his services in connection with the location and design of the New York Central Terminal at Buffalo were, perhaps, the crowning achievement of a distinguished career.

His interests and activities were many and varied, and in addition to his civil life, he had an honorable military record. He enlisted in the Sixty-fifth Infantry, New York National Guard, in 1889, advancing to Corporal, Sergeant, Second and First Lieutenant, and, in May, 1898, he received his Commission as Captain, in which rank he served throughout the Spanish-American War. He was placed on the Reserve List in 1918, re-assigned to the Sixty-fifth Field Artillery in 1919, and appointed Major of Field Artillery and placed on the Retired List in 1920.

Major Norton's services to the Buffalo Chamber of Commerce as Vice-President and as member of the Board of Directors for two consecutive terms,

were long and useful. He was particularly active on the City Planning and Marine Airport Committees, and developed the first plans for the landing area at the foot of Georgia Street, Buffalo.

He was the author of various articles for technical papers and social publications. He was a Past-President of the Buffalo Section of the American Society of Civil Engineers, the Engineering Society of Buffalo, and the American Society for Municipal Improvements. He was a member of the City Planning Institute and the Cornell Society of Engineers.

In civil and social organizations of the city Major Norton also was active. He was a member of many prominent groups, some of which are as follows: The Automobile Club of Buffalo, the Scottish Rite Masons, the Shrine, the Rotary Club, the Torch Club, the University Club, the Buffalo Athletic Club, and the Hook and Axe Club. He was a member of the Central Presbyterian Church.

Major Norton's life was cut short at the crest of his active career, when he passed away on March 4, 1930. He was one of the finest of men, a worker who loved his work and who sought no ostentatious praise. He was filled with the spirit of service to the community, and possessed a kindly, gentle character that endeared him to all who knew him. He was a highly esteemed gentleman, and, in his death, Buffalo has lost one of its great builders. He was deeply beloved by all and his passing leaves a void that can not easily be filled.

Fitting testimonials to Major Norton's sterling qualities have appeared in many publications in his own section of New York State, typical of which is the following from the *Buffalo Evening News:*

"The death of Major George H. Norton removes a man who has had a part in nearly every big public improvement that has been carried out in Buffalo during the last forty years. In many of them he was the directing mind. As City Engineer and, later, as Engineer for the Grade Crossing Commission, he had responsibilities far beyond any credit which his modest mind ever thought to claim.

"Moreover, it may be said for him that in a long lifetime of public employment, he retained the confidence and the friendship of those with whom he was associated to such a degree that even dissent from his plans never meant personal criticism. Closely associated as he was with government, he was regarded by everybody as distinctly an engineer, not a politician."

He is survived by his widow, Mrs. Elvina Margaret Schiferle Norton, and a son, George H. Norton, Jr.

Major Norton was elected a Member of the American Society of Civil Engineers on April 7, 1915, and at the time of his death was serving on the Board of Direction, his term of office not expiring until 1932.

HARRY TAYLOR, M. Am. Soc. C. E.*

Died January 27, 1930.

Harry Taylor, the son of John Franklin and Lydia Taylor, was born at Tilton, N. H., on June 26, 1862. His ancestors were of English stock who

* Memoir prepared by Herbert Deakyne, M. Am. Soc. C. E.

came to the United States about 1650 and were among the early settlers of Central New Hampshire. They helped found the Town of Sandbornton Bridge, later called Tilton, and several of them served in the Revolutionary War. Prior to becoming a Cadet at the United States Military Academy, West Point, N. Y., Harry Taylor resided in Tilton and attended the Tilton Academy. He entered the U. S. Military Academy on July 1, 1880, was graduated on June 15, 1884, and was appointed Second Lieutenant in the Corps of Engineers, U. S. Army.

Lieutenant Taylor's first assignment was at Willets Point (now Fort Totten), N. Y., where he served with the Battalion of Engineers and at the Engineer School of Application from 1884 to 1887. In 1887, he was assigned to duty at Wilmington, N. C., where he served as Assistant to the late Brigadier General (then Captain) William H. Bixby, Corps of Engineers, U. S. A., M. Am. Soc. C. E., the District Engineer in charge of river and harbor improvements in North Carolina and South Carolina. He was promoted to First Lieutenant on December 1, 1887. In 1888, he was transferred to duty at the U. S. Military Academy, where he remained about one year, being assigned in 1889 to duty under the orders of Lt.-Col. G. L. Gillespie, Corps of Engineers, U. S. A., then at New York, N. Y., and was placed in local charge of the construction of new fortifications at Sandy Hook, N. J. In 1891, Lieutenant Taylor was transferred to the Pacific Coast where he was placed in local charge of the construction of the lock at The Cascades, Oregon, on the Columbia River. He supervised the erection of the lock-gates, the largest that had been erected to that time, and designed the operating machinery for them as well as many of the masonry details.

He was promoted to Captain, Corps of Engineers, January 6, 1896, and in the same year was placed in charge of the construction of fortifications on Puget Sound and also of the improvement of waterways in the newly-created Seattle Engineer District, remaining on this duty until 1900. He was then assigned to duty at Boston, Mass., in charge of the construction of fortifications and the works of river and harbor improvement in the Boston Engineer District. He remained there until 1903. Captain Taylor was placed in command of Company L, 3d Battalion of Engineers, in 1903, and took his company to the Philippine Islands in October of that year. He served in the Philippines until October, 1905, and while there was on duty, successively, as Engineer Officer, Department of Luzon; Commander of the 3d Battalion of Engineers; and District Engineer in charge of fortification construction, which was initiated in 1904. He was promoted to the grade of Major, Corps of Engineers, in 1904.

Returning to the United States, Major Taylor, after a few months' duty in the Office of the Chief of Engineers, was assigned, in 1906, to duty at New London, Conn., in charge of river and harbor work in that vicinity and of fortification work at the eastern entrance of Long Island Sound. He was also charged with the purchase and issue of 60-in. searchlights and gasoline-actuated generating sets required for fortifications in the United States and its possessions. The purchasing agency established by him at New London was later moved to Washington, D. C., and during the World War was the pro-

curement agency which purchased a large part of the equipment and materials required by the Engineer Service. It still exists as part of the Supply Section of the Office of the Chief of Engineers.

Major Taylor was promoted to the grade of Lieutenant-Colonel, Corps of Engineers, June 24, 1909. He remained on duty at New London until 1911, when he was transferred to the Office of the Chief of Engineers, Washington, D. C., and was placed in charge of the River and Harbor Section of that office, where he remained until 1916. He reached the grade of Colonel, Corps of Engineers, February 28, 1915. In 1916 he was transferred to duty in New York City, in charge of the work of river and harbor improvement in the First New York District, and as Department Engineer of the Eastern Department, U. S. Army. At the time of Colonel Taylor's assignment to duty in New York, the prospect of the entry of the United States into the World War was growing, and, as a result, in addition to his regular duties, he was engaged in the examination of applicants for commissions in the Reserve Corps and in the selection of sites for camps and for the training of troops.

On May 26, 1917, Colonel Taylor was selected as Chief Engineer, American Expeditionary Forces; he sailed for France on May 28, 1917. He was appointed Brigadier-General, National Army, August 5, 1917, and remained in France until September 12, 1918, initiating the large engineering works required for the support of the U. S. Army in France.

On his return to the United States he was assigned to duty in the Office of the Chief of Engineers, Washington, D. C. He was honorably discharged as Brigadier General, June 1, 1919, and on July 1, 1920, was appointed Brigadier General and Assistant Chief of Engineers. General Taylor on his assignment to duty in the Office of the Chief of Engineers in 1918 was again placed in charge of the River and Harbor Section and continued in this work until his appointment as Major General, Chief of Engineers, on June 19, 1924. He served as Chief of Engineers until June 26, 1926, when he was placed on the retired list of the Army, having reached the statutory age of sixty-four years.

General Taylor was awarded the Distinguished Service Medal for his services in the American Expeditionary Forces with the following citation:

"For exceptionally meritorious and distinguished services. Arriving in France June 11, 1917, as chief engineer, American Expeditionary Forces, he organized and administered the Engineer Department, which included the construction of wharves, depots, railways, barracks, and shelters throughout the theater of operations. He continued these duties with most marked and conspicuous ability, building a complete and efficiently functioning institution."

He was also made a Commander of the Legion of Honor by the French Government for his services during the war.

During his forty-two years' service as an officer of the Corps of Engineers, General Taylor served on many official boards, both in connection with the preparation of plans for the defense of the coasts and in the interest of the improvement and utilization of the rivers and harbors of the United States

for navigation and other purposes. He was a member of an Advisory Board of Officers of the Corps of Engineers appointed in 1910 by direction of the President to assist the Secretary of the Interior in connection with granting permission to the City and County of San Francisco to use the Hetch Hetchy Valley in California for a new water supply system.

His experience in fortification construction began with the construction of new works of harbor defense in the early Nineties when Congress, realizing the defenseless condition of the coasts, again began to make appropriations for new fortification construction after a lapse of about twenty years. He was an outstanding fortification engineer, and when a Captain, devised with Capt. R. R. Raymond, Corps of Engineers (now Colonel, U. S. Army, (*Retired*)), an ingenious hoist for transferring the heavy ammunition from the magazine level of the batteries to the firing platform above. This hoist was used to a large extent in batteries constructed prior to the World War. During his long service in the Office of the Chief of Engineers as an Assistant to the Chief, Assistant Chief, and Chief of Engineers, General Taylor's duties were primarily in connection with the improvement and maintenance of rivers and harbors for the purpose of navigation. He acquired an extended acquaintance with the numerous projects adopted or under consideration by Congress, and his intimate knowledge of works of river and harbor improvement, combined with his sound judgment of their value to water-borne commerce, was of great assistance to the committees of Congress in the preparation of appropriation bills for these works. While serving in the Office of the Chief of Engineers he had supervision over many important engineering works for the Federal Government. One of these was the Wilson Dam on the Tennessee River at Muscle Shoals, Ala., in which he was particularly interested and which was completed while he was Chief of Engineers.

General Taylor was an engineer of great ability, with an active, logical mind, and his decisions were made quickly and with excellent judgment. Following his retirement from the Army in 1926, he engaged in consulting practice and was actively occupied with this work at the time of his death.

His youth in his native State had been spent largely in the open, and he had a fondness for hunting and fishing which remained with him all his life. This liking for outdoor life was no doubt largely responsible for his excellent physique, and for his physical appearance which gave little evidence of advancing years. His untimely death from pneumonia, after a brief illness, in Washington, D. C., where he had made his home after retirement, was a shock to his many friends. He was buried in the Arlington National Cemetery where he rests after a life that devoted to his country's service the talents which brought him honor and distinction.

General Taylor was married at Portsmouth, N. H., on October 30, 1901, to Adele Austin Yates, the daughter of Capt. Arthur Reid Yates, U. S. N., of Schenectady, N. Y., and Mrs. Yates, who was Susan Thompson Dwight, of Portsmouth. He is survived by Mrs. Taylor and their two children, Arthur Yates Taylor and Margaret Taylor, now Mrs. Alfred Craven Bruce.

He was a member of the Society of American Military Engineers, and served as its President in 1925. He was also a member of the Army and Navy and Chevy Chase Clubs of Washington.

General Taylor was elected a Member of the American Society of Civil Engineers on October 7, 1896.

MORGAN EDWARD YEATMAN, M. Am. Soc. C. E.*

Died November 17, 1929.

Morgan Edward Yeatman was born at East Sheen, Surrey, England, on August 8, 1851, the son of Morgan and Mary C. L. (Penrhyn) Yeatman. He was educated at Marlborough, graduated with honors in Mathematics and Classics at Trinity College, Cambridge, became Wrangler in 1874, and Master of Arts in 1877.

Mr. Yeatman was articled in 1874 to George Fosbery Lyster, Chief Engineer to the Mersey Docks and Harbour Board and was engaged on extensive new dock and harbor works for that Board, including the preparation of the first plans for the Liverpool Overhead Railway from 1877 to 1881.

He then came to the United States and served until 1884 as Assistant Engineer on the construction of the Pittsburgh, McKeesport, and Youghiogheny Railroad, under the late Jonathan Wainwright, M. Am. Soc. C. E., Chief Engineer. Mr. Yeatman was engaged on the design and construction of many of the bridges on that road.

From 1884 to 1885, he was Principal Assistant Engineer (Maintenance) on the Pittsburgh and Lake Erie Railroad (Lessees of the Pittsburgh, McKeesport, and Youghiogheny Railroad). In 1886, with Mr. Wainwright, he entered into private practice as Consulting and Constructing Engineer with headquarters in Pittsburgh, Pa. In this capacity he was engaged until 1888 on various bridges and railroads, his experience including bridge detailing for the Keystone Bridge Works and the Iron City Bridge Works (C. J. Schultz and Company).

From 1889 to 1896, Mr. Yeatman was Principal Assistant Engineer, Maintenance of Way, on the Norfolk and Western Railroad, with headquarters at Roanoke, Va., in charge of bridge work on 1 500 miles of line. This work included the renewal of a large number of the older bridges. He compiled the first Standard Bridge Specification for this railroad.

From 1898 to 1908 he was on the staff of S. Pearson and Son, on the design of new bridges for the Tehuantepec National Railroad, in Mexico, and the Arica-La Paz Railway in Chile. He also designed the structural steelwork for the Mexican Ports of Coatzacoalcos and Salina Cruz, including wharves, quay walls, breakwaters, and other harbor works, drainage works, and water supply.

In 1908, Mr. Yeatman began the private practice of engineering in England, which practice he continued in Westminster until 1915. He was a

* Memoir prepared by S. Alfred Frech, Cons. Civ. Engr., London, S. W. 1, England.

Member of the Institution of Civil Engineers. During the World War he gave his services to the Mechanical Warfare Department, and was engaged chiefly in the design of "tanks", until the Armistice was signed in 1918.

In 1888, at Pittsburgh, he was married to Blanche D. Fullerton, and is survived by his widow and three children—two sons, Morgan John, who is Lieutenant-Commander in the Royal Navy, and Edward Stanley; and one daughter, Ellinor (Mrs. C. E. May).

He was a man of many hobbies; he was fond of sailing and was an exceptionally fine musician. By his death, which occurred at his residence at Hill House, Buckingham, on November 17, 1929, after a short illness, at the advanced age of 78 years, the Engineering Profession loses a man of keen intellect, a great mathematician, and bridge expert.

Mr. Yeatman was elected a Member of the American Society of Civil Engineers on May 14, 1892.